Iowa Calibration of MEPDG Performance Prediction Models

Final Report
June 2013

IOWA STATE UNIVERSITY
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16. Abstract This study aims to improve the accuracy of AASHTO Mechanistic-Empirical Pavement Design Guide (MEPDG) pavement performance predictions for Iowa pavement systems through local calibration of MEPDG prediction models. A total of 130 representative pavement sites across Iowa were selected. The selected pavement sites represent flexible, rigid, and composite pavement systems throughout Iowa. The required MEPDG inputs and the historical performance data for the selected sites were extracted from a variety of sources. The accuracy of the nationally-calibrated MEPDG prediction models for Iowa conditions was evaluated. The local calibration factors of MEPDG performance prediction models were identified to improve the accuracy of model predictions. The identified local calibration coefficients are presented with other significant findings and recommendations for use in MEPDG/DARWin-ME for Iowa pavement systems.			
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IOWA CALIBRATION OF MEPDG PERFORMANCE PREDICTION MODELS

Final Report
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EXECUTIVE SUMMARY

The latest AASHTOWare® pavement design software, DARWin-ME™ (AASHTO 2011), and AASHTO Mechanistic-Empirical Pavement Design Guide (MEPDG) (AASHTO 2008) are significantly improved methodologies for the analysis and design of pavement structures. The DARWin-ME™ builds upon the National Cooperative Highway Research Program (NCHRP) 1-37A project (NCHRP 2004), MEPDG, and the associated research-grade software version (MEPDG version 1.1).

The national calibration-validation process was successfully completed for MEPDG (NCHRP 2004). Although this effort was comprehensive, further calibration and validation studies in accordance with local conditions are highly recommended by the MEPDG as a prudent step in implementing a new design procedure that is so different from the current procedures.

This research aims to improve the accuracy of MEPDG projected pavement performance predictions for Iowa pavement systems through local calibration of the prediction models. A total of 35 representative JPCP sections (rigid pavements), a total of 35 representative HMA sections (flexible pavements), and a total of 60 representative HMA over JPCP sections (composite pavements) were selected for this study. The required MEPDG inputs for the selected sections were collected primarily from the Iowa DOT pavement management information system (PMIS), material testing records and previous project reports relevant to MEPDG implementation in Iowa. A database of historical performance data for the selected sections was extracted from the Iowa DOT PMIS. The accuracy of the nationally calibrated MEPDG prediction models for Iowa conditions was evaluated. The local calibration factors of MEPDG prediction models were identified by using linear and nonlinear optimization procedures to improve the accuracy of model predictions.

The local calibration coefficients identified in this study are presented in Table 4 for JPCP, Table 5 for HMA pavement, and Table 6 for HMA over JPCP. The key findings from this study are:

- The locally calibrated faulting, transverse cracking, and IRI models for Iowa JPCP provide better predictions than their nationally calibrated counterparts.
- The identified local calibration factors increase the accuracy of rutting predictions and, to a lesser extent, longitudinal (top-down) cracking predictions for both Iowa HMA and Iowa HMA over JPCP.
- The nationally calibrated alligator (bottom-up) cracking model provides acceptable predictions for new Iowa HMA pavement.
- Both nationally and locally calibrated alligator (bottom-up) cracking models provide acceptable predictions for Iowa HMA over JPCP.
- Little or no thermal cracking is predicted when using the proper binder grade for Iowa climatic conditions, but significant thermal cracking is observed in both Iowa HMA and HMA over JPCP.
- Transverse cracking records in Iowa DOT PMIS do not differentiate thermal cracking and reflection cracking measurements for HMA over JPCP.

- Good agreement is observed between the IRI measures for Iowa HMA pavement and HMA over JPCP and the MEPDG predictions from: (1) IRI model of nationally calibrated distress inputs with national calibrated coefficients and (2) IRI model of locally calibrated distress inputs with national calibrated coefficients.

Future recommendations for use of MEPDG/DARWin-ME^{TM} in Iowa pavement systems include:

- The locally calibrated JPCP performance models (faulting, transverse cracking and IRI) identified in this study are recommended for use in Iowa as alternative to the nationally calibrated ones.
- The locally calibrated rutting prediction models identified in this study are recommended for use in HMA and HMA over JPCP systems as alternative to the nationally calibrated ones.
- The nationally calibrated alligator (bottom-up) cracking prediction models are recommended for use in Iowa HMA and HMA over JPCP systems.
- The use of MEPDG for longitudinal cracking, thermal cracking, and reflection cracking analysis in HMA and HMA overlay JPCP is recommended only for research investigations and not for routine decision making until these distress models are fully implemented.
- The use of national calibration coefficient of IRI models in Iowa HMA and HMA over JPCP systems is recommended because longitudinal cracking and thermal cracking models as IRI design inputs are still evolving and the accuracy of national calibrated IRI model is acceptable for Iowa conditions.
- Preliminary studies were carried out to see if there are any differences between the latest MEPDG (version 1.1) and DARWin-ME^{TM} performance predictions for new JPCP, new HMA, and HMA over JPCP. The results indicated that the differences between the predictions of the two software versions are quite significant, at least in some cases, warranting further investigation to determine if the local calibration study needs to be repeated using the DARWin-ME^{TM} solution, which is now referred to as AASHTOWare Pavement ME Design (version 1.3).

INTRODUCTION

The latest AASHTOWare® pavement design software, DARWin-ME™ (AASHTO 2011), and the AASHTO Mechanistic-Empirical Pavement Design Guide (MEPDG) (AASHTO 2008) are significantly improved methodologies for the analysis and design of pavement structures. The DARWin-ME™ builds upon the National Cooperative Highway Research Program (NCHRP) 1-37A project (NCHRP 2004) on the development of the MEPDG and the associated research-grade software (version 1.1).

The mechanistic part of MEPDG is the application of the principles of engineering mechanics to calculate pavement responses (stresses, strains, and deflection) under loads for the predictions of the pavement performance history. The empirical nature of the MEPDG stems from the fact that the laboratory-developed pavement performance models are adjusted or calibrated to the observed performance measurements (distresses) from the actual pavements. Clearly, the MEPDG's mechanistic-empirical procedure will require significant effort to successfully implement a useful design procedure. Without calibration to actual pavement performance measures, the results of mechanistic calculations cannot be used to predict rutting, cracking, and faulting with any degree of confidence. The distress mechanisms are far more complex than can be practically modeled; therefore, the use of empirical factors and calibration is necessary to obtain realistic performance predictions.

The MEPDG does not provide a design thickness as the end product. Instead, it provides the pavement performance throughout its design life. The design thickness can be determined by modifying the design inputs and obtaining the best performance with an iterative procedure. The performance models used in the MEPDG are nationally calibrated using design inputs and performance data largely from the national Long-Term Pavement Performance (LTPP) database. Especially, the LTPP database used for national (global) calibration of MEPDG includes no Hot Mix Asphalt (HMA) pavement sections, but only one Portland Cement Concrete (PCC) pavement section in Iowa (NCHRP 2004). Also, a previously completed research study by the authors (Kim et al. 2010) in pursuit of the MEPDG implantation initiatives in Iowa indicated the need for local calibration of MEPDG performance prediction models for Iowa conditions. Thus, it is necessary to calibrate the MEPDG performance models for local highway agencies' implementation by taking into account local materials, traffic information, and environmental conditions.

OBJECTIVES

The primary objective of this research is to improve the accuracy of MEPDG projected pavement performance predictions for Iowa pavement systems through local calibration of MEPDG performance prediction models.

SUMMARY OF LITERATURE REVIEW RESULTS

The national calibration-validation process was successfully completed for MEPDG (NCHRP 2004). Although this effort was comprehensive, further calibration and validation studies to suit local conditions are highly recommended by the MEPDG as a prudent step in implementing a new design procedure that is so different from the current procedures. Several national-level research studies supported by the NCHRP and FHWA have been conducted after the release of the original research version of the MEPDG software. Parallel to national-level research projects, many state/local agencies have conducted or plan to undertake local calibration studies for their own pavement conditions.

There are two NCHRP research projects that are closely related to local calibration of MEPDG performance predictions. They are: (1) NCHRP 9-30 project (NCHRP 2003a, NCHRP 2003b), "*Experimental Plan for Calibration and Validation of Hot Mix Asphalt Performance Models for Mix and Structural Design"*, and (2) NCHRP 1-40B (Von Quintus et al. 2005, NCHRP 2007, Von Quintus et al. 2009a, Von Quintus et al. 2009b, NCHRP 2009, TRB 2010), "*User Manual and Local Calibration Guide for the Mechanistic-Empirical Pavement Design Guide and Software*". Under the NCHRP 9-30 project, pre-implementation studies involving verification and recalibration have been conducted in order to quantify the bias and residual error of the flexible pavement distress models included in the MEPDG (Muthadi 2007).

Based on the findings from the NCHRP 9-30 study, the NCHRP 1-40B project has focused on preparing (1) a user manual for the MEPDG and its software, and (2) detailed, practical guide for highway agencies for local or regional calibration of the distress models in the MEPDG and its software. The manual and guide have been presented in the form of draft AASHTO recommended practices with two or more examples or case studies illustrating the step-by-step procedures. It was also noted that the longitudinal cracking and reflection cracking models was not much considered in the local calibration guide development during NCHRP 1-40B study due to lack of accuracy in the predictions (Muthadi 2007,Von Quintus and Moulthrop 2007). The NCHRP 1-40 B was completed in 2009 and is now published under the title, "Guide for the Local Calibration of the Mechanistic-Empirical Pavement Design Guide" through AASHTO (AASHTO 2010).

Two research studies supported by the FHWA have been conducted to use pavement management information system (PMIS) data for local calibration of MEPDG. The study on "*Using Pavement Management Data to Calibrate and Validate the New MEPDG, An Eight State Study"* (FHWA 2006a, FHWA 2006b) evaluated the potential use of PMIS for MEPDG local calibration. Eight States participated in this study: Florida, Kansas, Minnesota, Mississippi, New Mexico, North Carolina, Pennsylvania, and Washington. The study concluded that all the participating States could feasibly use PMIS data for MEPDG calibrations and others States not participating in this study could also do the same. It was further recommended that each SHA should develop a satellite pavement management/pavement design database for each project being designed and constructed using the MEPDG as part of the currently used PMIS.

The second follow-up study, FHWA HIF-11-026, "*the local calibration of MEPDG using* *pavement management system"* (FHWA 2010a, FHWA 2010b) was conducted to develop a framework for using existing PMIS to calibrate the MEPDG performance models. One State (North Carolina) was selected based on screening criteria to finalize and verify the MEPDG calibration framework based on the set of actual conditions. Following the developed framework, local calibration for the selected State was demonstrated under the assumptions of both MEPDG performance predictions established from NCHRP 1-37 A as well as distress measurements from a selected State.

Local/State level research studies have also been conducted apart from national-level research studies. Most studies focused on flexible pavements and a few studies conducted for rigid pavements primarily focused on jointed plain concrete pavement (JPCP). Flexible pavement calibration studies, including new HMA pavement and HMA overlaid pavements, include the work by Galal and Chehab (2005) in Indiana; Von Quintus and Moulthrop (2007) in Montana; Kang et al. (2007) mainly in Wisconsin; Schram and Abdelrahman (2006) in Nebraska; Muthadi and Kim (2008), Corley-Lay et al. (2010), and Jadoun (2011) in North Carolina; Li et al. (2009) and Liu et al. (2010) in Washington; Banerjee et al. (2009), Banerjee et al. (2010), and Banerjee et al. (2011) in Texas; Glover and Mallela (2009) in Ohio; Darter et al (2009) in Utah; Souliman et al. (2010) and Mamlouk and Zapata (2010) in Arizona; Kim et al. (2010) in Iowa; Khazanovich et al. (2008), Velasquez et al (2009) and Hoegh et al. (2010) in Minnesota; and Hall et al (2011) in Arkansas.

Limited studies on rigid pavement performance prediction model calibration, primarily focusing on JPCP, include the work by Li et al. (2006) in Washington; Schram and Abdelrahman (2006) in Nebraska; Darter et al. (2009) in Utah; Velasquez et al (2009) in Minnesota; Kim et al. (2010) in Iowa; Bustos et al. (2009) in Argentina; and Delgadillo et al (2011) in Chile. The procedures and findings of all these studies related to both flexible and rigid pavements are summarized in Appendix A. Several significant issues that are relevant to the present study are highlighted below:

1. None of the State-level studies covered all pavement types and performance measures in their MEPDG local calibration efforts. Even the national-level studies did not recalculate calibration factors of all performance models because of the good accuracy of nationally calibrated model predictions, continuous improvement of performance models with time, and lack of field measurements.

2. Rutting, alligator (bottom-up) cracking, and IRI predictions for flexible pavement could be improved through local calibration. Some State-level studies conducted in Minnesota and Montana reported that MEPDG over-predicts total rut depth because significant rutting was predicted in unbound layers and embankment soils.

3. No consistent trend in the longitudinal (top-down) cracking predictions of flexible pavement could be identified to reduce the bias and standard error, and improve the accuracy of this prediction model.

4. Few or no thermal (transverse) cracking is predicted by MEPDG when using a properly selected PG binder for local conditions. However, transverse cracking is in fact observed in actual HMA pavement.

5. None of the studies attempted to calibrate the current empirical reflection cracking model of HMA overlaid pavement in MEPDG.

6. Limited local calibration studies for JPCP indicated that faulting, transverse cracking and IRI could be improved by local calibration.

CALIBRATION METHODOLOGY

Based on literature review, a set of procedures for local calibration of the MEPDG performance predictions was made in consultation with the Iowa DOT engineers. The procedure is detailed into the following steps.

Step 1: Select typical pavement sections around the State
Step 2: Identify available sources to gather input data and determine the desired level for obtaining each input data
Step 3: Prepare MEPDG input database from available sources including Iowa DOT PMIS, material testing records, design database, and research project reports relevant to MEPDG implementation in Iowa
Step 4: Prepare a database of performance data for the selected Iowa pavement sections from Iowa DOT PMIS
Step 5: Assessment of local bias from national calibration factors
Step 6: Identification of local calibration factors (sensitivity analysis and optimization of calibration factors)
Step 7: Determination of adequacy of local calibration factors

Site Selection

To develop the database for MEPDG local calibration, representative pavement sites across Iowa were selected in consultation with Iowa DOT engineers with the following considerations:

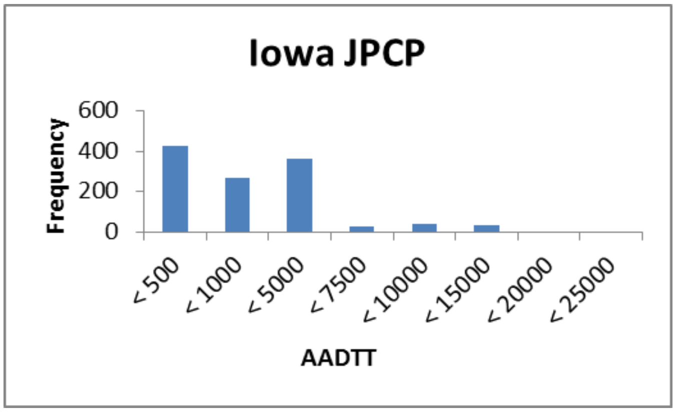
- Different pavement types (rigid, flexible, and composite)
- Different geographical locations
- Different traffic levels

Table 1 lists the number of pavement sections selected for this study. A total of 35 sections for new JPCP (rigid pavements), a total of 35 sections for new HMA pavements (flexible pavements), and a total of 60 sections for HMA over JPCP (composite pavements) were selected from a list of potential roadway segments for each pavement type identified. Note that the list of potential roadway segments includes 125 for JPCP (small set of 60 for JPCP), 35 for HMA, and 85 for HMA over JPCP. In the selected new JPCP and new HMA roadway segments, twentyfive sections were utilized for calibration and 10 sections were utilized for verification of identified calibration coefficients. In the selected HMA over JPCP roadway segments, forty-five sections were utilized for calibration and 15 sections were utilized for verification of identified calibration coefficients. Although a total of 75 pavement sections were initially proposed for analysis, the final test matrix included a total of 130 pavement sections to accommodate a variety of pavement structures typically found in Iowa especially for composite pavements.

Table 1. Site selection summary information

Type	Iowa PMIS Code	Number of Sites Selected	Iowa LTPP sections
JPCP	1	35	6
HMA	4	35	1
HMA over JPCP	3 and 3A	60	9

Figure 1 presents the average annual daily truck traffic (AADTT) distributions for each type of Iowa pavements. As seen in this figure, HMA surface pavements are more used with lower AADTT while JPCPs are more used with higher AADTT. To comprise all traffic conditions found in Iowa, three categories of traffic levels were utilized in selecting sites for calibration. AADTT fewer than 500 is categorized as low traffic volume, anywhere between 500 and 1,000 is categorized as medium traffic volume, and AADTT higher than 1,000 is categorized as high traffic volume. The selected sections also represent a variety of geographical locations across Iowa as seen in Figure 2.



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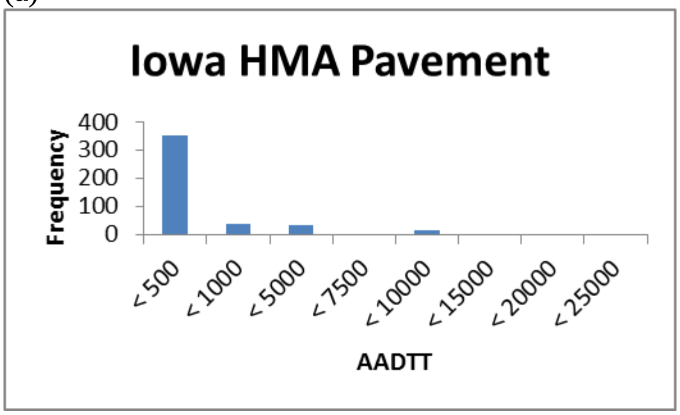

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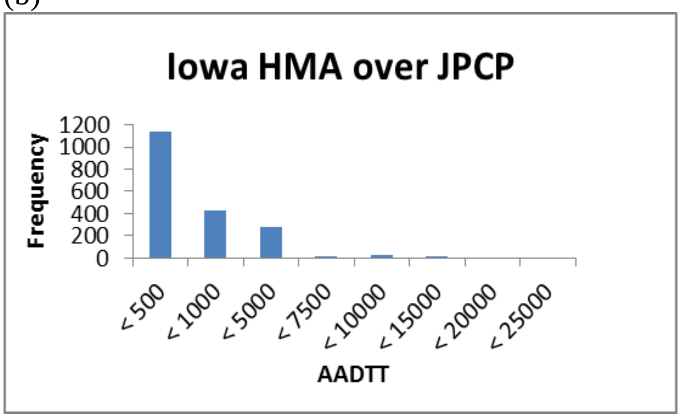

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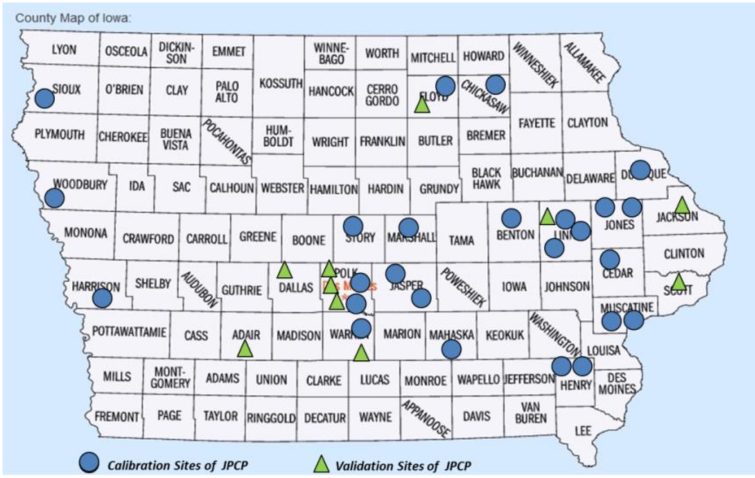
Figure 1. Iowa pavements by AADTT distribution as of 2011: (a) JPCP, (b) HMA pavement, and (c) HMA over JPCP



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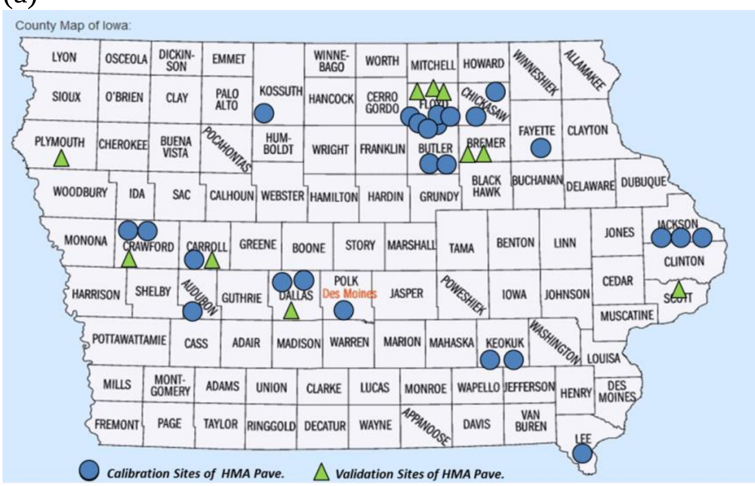

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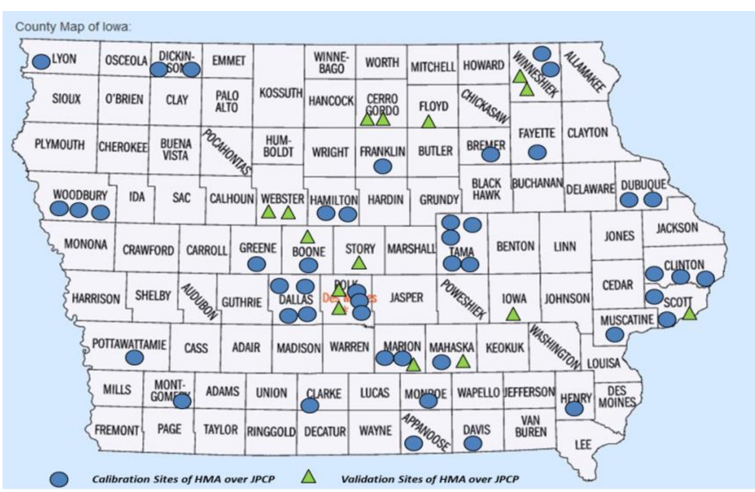


(c)

Figure 2. Geographical locations of selected Iowa pavement sites: (a) JPCP, (b) HMA pavement, and (c) HMA over JPCP

MEPDG Calibration Database

MEPDG Input Database

The MEPDG inputs required for the selected sections were primarily obtained from the Iowa DOT PMIS and material testing records. Other major sources of the data include online project reports relevant to MEPDG implementation in Iowa (http://www.iowadot.gov/operationsresearch/reports.aspx; http://www.ctre.iastate.edu/research/reports.cfm).

If a specific input data was not available, the default value or its best estimate was inputted considering its level of sensitivity with respect to MEPDG predicted performance. The NCHRP 1-47 project final report, "*Sensitivity Evaluation of MEPDG Performance Prediction*," was referred to assess the level of MEPDG design input sensitivity. The NCHRP 1-47 project report documents most of the MEPDG sensitivity studies conducted up to date using the initial version to the latest version of the MEPDG software. It also presents results of most comprehensive MEPDG sensitivity analyses (local and global sensitivity analyses) carried out through this project under five climatic conditions and three traffic levels in the US (Schwartz et al 2012).

A detailed database was prepared and formatted in a manner suitable for input to the MEPDG software. The descriptions of the input data and sources are presented at length below.

General Project Inputs

The general project inputs section of the MEPDG is categorized into general information, site/project identification information, and the analysis parameters. General information consists of information about the pavement type, design life, and the time of construction. Site/project identification information includes pavement location and construction project identification. The analysis parameters require initial smoothness (IRI), distress limit criteria and reliability values. Most of this information in the general project inputs section can be obtained from Iowa DOT's PMIS. The MEPDG default values were applied to distress limit criteria.

Traffic Inputs

The base year for the traffic inputs is defined as the first calendar year that the roadway segment under design is opened to traffic. Four basic types of traffic data for the base year are required by the MEPDG: (1) Traffic volume, (2) Traffic volume adjustment factors, (3) Axle load distribution factors, and (4) General traffic inputs. Iowa DOT's PMIS provides annual average daily truck traffic (AADTT) at base year under traffic volume and traffic speed limit. Google Maps (http://maps.google.com/) was utilized to observe the detailed roadway features including the number of lanes per traffic direction and types of shoulder.

Other traffic input data required by the MEPDG were not available in both Iowa DOT's PMIS and previous project reports. Most of these traffic input data are required to project further detailed load spectra. It is clear that higher AADTT per design lane translates into increased number of applied load repetitions on pavement resulting in increased pavement responses and distress predictions. However, a strict requirement of detailed load spectra in the current state-of-the-art pavement design is open for debate (Bordelon et al. 2010) although the use of load spectra seems more rational and an accurate approach to characterize traffic properties (Tam and Von Quintus 2003). Thus, the default MEPDG values or the values recommended by NCHRP 1-47A reports were used in place of the other unavailable traffic input values.

Climate Inputs

The MEPDG software includes climate data at weather stations in each State. The MEPDG software can also generate climate data by extrapolating nearby weather stations if the latitude and longitude are known. The specific location information of selected sections obtained from Iowa DOT PMIS was inputted and then the climate data of each section was generated.

Pavement Structure Inputs

The MEPDG pavement structure inputs include types of layer material and thicknesses. This information can be obtained from Iowa DOT PMIS. For selected HMA over JPCP sections, the percent of slab cracked after repair was needed to estimate the existing JPCP structural capacity. A 15% JPCP cracking design limit was used for this required input under the assumption that design limit is the trigger value for rehabilitation of existing JPCP.

Material Property Inputs

The task of obtaining detailed material properties, especially for older pavements, from available resources was not easy. It was also difficult to ascertain if the MEPDG default values are applicable to Iowa conditions. Previous project reports related to MEPDG implementation in Iowa were reviewed. Typical PCC materials properties for Iowa pavements were obtained from the final report on CTRE Project 06-270, "*Iowa MEPDG Work Plan Task 4: Testing Iowa Portland Cement Concrete Mixtures for the AASHTO Mechanistic-Empirical Pavement Design Procedure*" (Wang et al. 2008a).

For HMA materials properties in Iowa, an Iowa DOT HMA mix design database containing more than 4000 construction projects was reviewed and utilized. In cases where the HMA mix design information was not available for specific sections selected, the asphalt binder grade was determined from the LTPPBind program and the typical aggregate gradation of Iowa HMA mixture was obtained by averaging HMA aggregate gradation reported in the Iowa DOT HMA mix design database. Note that the asphalt binder properties dominate the HMA dynamic modulus prediction models used in MEPDG levels 2 and 3 analyses (Schwartz 2005, Ceylan et al. 2009) and is therefore more sensitive to HMA pavement performance predictions (Schwartz et al 2011).

Typical thermal properties of HMA and PCC in Iowa can be obtained from final report on CTRE Project 06-272, "*Iowa MEPDG Work Plan Task 6: Material Thermal Input for Iowa Materials*" (Wang et al. 2008b). Typical Iowa soil and aggregate properties can be extracted from final report on "*Iowa MEPDG Work Plan Task 5: Characterization of Unbound Materials (Solis/Aggregates) for Mechanistic-Empirical Pavement Design Guide*", which is set to be released soon.

Pavement Distress Database

A database of historical performance data for the selected sections was prepared from Iowa DOT PMIS. Most of the MEPDG performance predictions are recorded in Iowa DOT PMIS. However, some differences between PMIS distress measures and MEPDG performance predictions were identified. For calibration of the MEPDG performance prediction models, the identified differences were resolved by taking into account the following assumptions:

- MEPDG provides rutting predictions for individual pavement layers while Iowa DOT PMIS provides only accumulated (total) rutting observed in HMA surface. Rutting measurements for individual layers were computed by applying average percentage of total rutting for different pavement layers and subgrade recommended in the NCHRP 1-37A report (NCHRP 2004) on HMA surface rutting recorded in Iowa DOT PMIS.
- MEPDG transverse cracking predictions for new HMA and HMA overlaid pavements are considered as thermal cracking. The PMIS transverse cracking measurements for new HMA pavement could be considered as HMA thermal cracking, but the ones recorded for HMA overlaid pavements could either be reflection cracking or thermal cracking. However, transverse cracking measurements in Iowa DOT PMIS for HMA overlaid pavements were not differentiated as such. The reflection cracking model implemented in the current MEPDG is purely empirical and the M-E based reflection cracking models developed through the NCHRP 1-41 project (Lytton et al. 2010) are considered to be added in future refinement of DARWin-ME™. Therefore, the calibration of current empirical reflection cracking model in MEPDG was not considered in this study.
- The units reported in PMIS for the JPCP transverse cracking and the alligator and thermal (transverse) cracking of HMA and HMA overlaid pavements are different from those used in MEPDG. These distress measured values in PMIS are converted into same units of those MEPDG predictions in accordance to AASHTO guide for the local calibration of the MEPDG (2010)
- Some irregularities in distress measures were identified in Iowa DOT PMIS. Occasionally, distress magnitudes appear to decrease with time or show erratic patterns without explanation. In these cases, the distress measure history curves were refined not to decrease with time.

Identification of Local Calibration Factors

Figure 3 depicts the procedure to identify local calibration factors (coefficients) of MEPDG performance prediction models in this study. As a first step, sensitivity analyses of calibration coefficients on MEPDG predictions were performed. Two optimization approaches were utilized depending on the constitution of MEPDG performance prediction models. More details are presented in the following subsections.

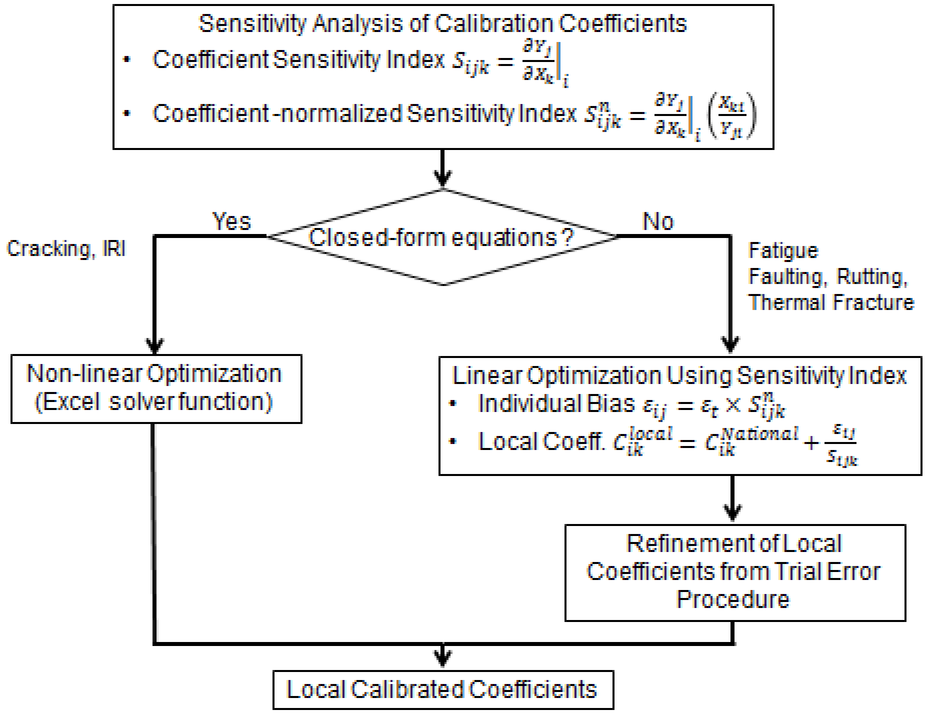


Figure 3. Flow chart of the procedure used in determination of local calibration factors

Sensitivity Analysis of MEPDG Performance Prediction Model Calibration Coefficients

Sensitivity analysis (SA) is the apportionment of output variability from a model to its various inputs. Sensitivity of MEPDG performance predictions to calibration coefficients was analyzed to: (1) to derive a better understanding of how the values of calibration coefficients affect performance predictions, and (2) to reduce the search space for subsequent calibration coefficient optimization by identifying the changes in performance predictions to changes in calibration coefficients. A coefficient sensitivity index (S_{ijk}) and a coefficient- normalized sensitivity index (S^n_{ijk}) were adapted to quantify the sensitivity of each calibration coefficient and to compare the sensitivity level among all calibration coefficients, respectively. The coefficient sensitivity index S_{ijk} is defined as:

$$S_{ijk} = \frac{\partial Y_j}{\partial X_k}\bigg|_i \cong \frac{\Delta Y_j}{\Delta X_k}\bigg|_i \qquad (1)$$

$$\frac{\Delta Y_j}{\Delta X_k}\bigg|_i = \frac{Y_{j,i+1} - Y_{j,i}}{X_{k,i+1} - X_{k,i}} \qquad \text{when } X_{j,i+1} > X_{j,i} \qquad (2)$$

$$\frac{\Delta Y_j}{\Delta X_k}\bigg|_i = \frac{Y_{j,i} - Y_{j,i-1}}{X_{k,i} - X_{k,i-1}} \qquad \text{when } X_{j,i-1} < X_{j,i} \qquad (3)$$

in which Y_{ji}, X_{ki} are the values of the performance prediction *j* and calibration coefficient *k* evaluated at national calibration coefficient condition *i* in a given performance prediction model. The partial derivative can be approximated using a standard central difference approximation. The S_{ijk} can be interpreted as the percentage change in performance prediction Y_j caused by a given percentage change in the calibration coefficient X_k at national calibrated condition *i* in a performance prediction model. For example, S_{ijk} = 0.5 implies that a 20% change in the calibration coefficient value of X_{ki} will cause a 10% change in performance prediction Y_{ji}. Two coefficient sensitivity indices (S_{ijk}) for each calibration coefficient X_k were calculated when increasing and decreasing the calibration coefficient values from national calibration coefficient value ($X_{j,i+1}>X_{j,i}$ and $X_{j,i-1}<X_{j,i}$). Since calibration coefficients at the national calibration condition *i* ranged broadly, they should have some scale for comparisons. Thus, S_{ijk} was normalized using the associated national calibration coefficient. A "national coefficient" normalized sensitivity index (S^n_{ijk}) was defined as:

$$S^n_{ijk} = \frac{\partial Y_j}{\partial X_k}\bigg|_i \left(\frac{X_{ki}}{Y_{ji}}\right) \cong \frac{\Delta Y_j}{\Delta X_k}\bigg|_i \left(\frac{X_{ki}}{Y_{ji}}\right) \qquad (4)$$

Two in-service pavements representing typical Iowa pavements were modeled for SA. These include a JPCP section in I-29, Harrison County, and a HMA section in US 61, Lee County. The modeled JPCP section consisted of 12-in thick PCC slab with 20-ft transverse joint spacing over a 10-in A-1-b granular base, and an A-7-6 compacted embankment subgrade. The modeled HMA pavement section consisted of 11-in thick HMA (PG 64-22 binder grade) over a 10-in A-1-b granular base, and an A-7-6 compacted embankment subgrade. AADTT values of 3,104 and 891 were inputted for the JPCP and the HMA pavement, respectively. The MEPDG climate files for these pavement locations were generated and inputted. The required other design inputs were prepared as described in the previous section on MEPDG input database preparation.

The nationally calibrated MEPDG performance model predictions for the JPCP resulted in 92% transverse cracking, 0.033-in faulting and 306-in/mile IRI for a 30-year design life. The nationally calibrated coefficients were utilized as base cases. The nationally calibrated performance model predictions for the HMA resulted in 0.15-in HMA rutting, 0.38-in total rutting, 0.1% alligator cracking, 0-ft/mile longitudinal cracking, 1-ft/mile thermal cracking, and 105.4-in/mile IRI for a 20-year design life. Note that the coefficients were varied between 20% and 50% of the nationally-calibrated coefficient values.

Table 2 and Table 3 summarize calibration coefficient sensitivity indices for the modeled JPCP and HMA pavements. The negative sign of the coefficient sensitivity indices means that performance predictions decrease with increase in calibration coefficients and vice versa.

Most calibration coefficients of the JPCP faulting prediction model except C7 affect the faulting predictions. For JPCP transverse cracking predictions, the fatigue model related calibration coefficients are the ones which are most sensitive in the transfer function. Note that the transfer function in transverse cracking models convert predicted fatigue damage from fatigue model to equivalent transverse cracking measurements. In the JPCP IRI models, coefficients C1 related to faulting and C4 related to site factors are the ones that are most sensitive.

Table 2. Summary of calibration coefficient sensitivity indices for JPCP

Distress	Coefficient	Coefficient sensitivity index (S_{ijk})		Coefficient - normalized sensitivity index (S^n_{ijk})
		$X_{j,i+1}>X_{j,i}$	$X_{j,i-1} < X_{j,i}$	
Faulting	C1	0.04	0.03	1.09
	C2	0.03	0.03	0.76
	C3	9.15	10.98	0.67
	C4	6.79	9.05	0.21
	C5	0.00	0.00	0.06
	C6	0.25	0.10	2.09
	C7	0.00	0.00	0.00
	C8	0.00	0.00	0.09
Fatigue for Crack	C1	-196.50	-19.75	-2.35
	C2	-299.18	-31.97	-2.20
Crack	C4	-7.50	-7.50	-0.08
	C5	-7.58	-11.11	0.20
IRI	C1	91.92	91.92	0.43
	C2	6.79	7.24	0.02
	C3	8.57	8.44	0.07
	C4	3.30	3.29	0.48

The rutting predictions are affected by most calibration coefficients in the HMA rutting prediction model. Similar to JPCP, the fatigue model related calibration coefficients in HMA longitudinal and alligator cracking model transfer functions are the most sensitive ones. The artificially large absolute values of coefficient-normalized sensitivity index (S^n_{ijk}) in B2 and B3 of fatigue model are related to near 0% longitudinal and alligator cracking predictions for the base cases. However, it can still be interpreted that B2 and B3 of fatigue model are more sensitive than B1. In HMA IRI models, coefficients C4 related to site factors and C1 related to rutting are the most sensitive ones compared to other coefficients.

The sensitivity results related to calibration coefficients in this study were made from limited sensitivity analysis using the local SA method. The much more computationally intensive global sensitivity analysis should be carried out to confirm these results. However, the local SA can still provide some insights into the sensitivity of MEPDG performance predictions to calibration coefficients to fulfill the objectives of this study.

Table 3. Summary of calibration coefficient sensitivity indices for HMA pavements

Distress	Coefficient	Coefficient sensitivity index (S_{ijk})		Coefficient - normalized sensitivity index (S^n_{ijk})
		$X_{j,i+1}>X_{j,i}$	$X_{j,i-1} < X_{j,i}$	
HMA Rut	B1	0.15	0.15	1.00
	B2	6.30	0.30	21.68
	B3	5.74	0.29	19.80
GB Rut	B1_Granular	0.04	0.04	1.00
SG Rut	B1_Fine-grain	0.19	0.19	1.00
Fatigue for LCrack	B1	-0.02	-0.02	-2.00
	B2	-0.02	-21,199	-1,060,000
	B3	4,199	0.02	210,000
Fatigue for ACrack	B1	-0.07	-0.23	-1.43
	B2	-0.21	-199.79	-943.40
	B3	182.79	0.21	863.21
LCrack	C1_Top	0.00	-0.06	-23.00
	C2_Top	-0.01	-0.13	-23.00
	C4_Top	0.00	0.00	1.00
ACrack	C1_Bottom	-0.19	-2.21	-11.32
	C2_Bottom	-0.13	-0.35	-2.29
	C4_Bottom	0.00	0.00	1.00
TCrack	K_Level 3	0.00	0.00	0.00
IRI	C1	0.38	0.39	0.15
	C2	0.00	0.00	0.00
	C3	0.00	0.00	0.00
	C4	2,293	2,000	0.31

Optimization Approaches

Nonlinear programming optimization technique through the MS Excel® solver routine has been commonly used to minimize the bias (ε) and the root mean square error (RMSE) between the actual distress measurements and the MEPDG predicted values (Velasquez et al. 2009, FHWA 2010a, Jadoun 2011). To use this approach, all input values required by the performance models are needed to satisfy closed form solution requirements. As seen in Figure 3, it was checked whether MEPDG could provide this information as well as the model input values required at output files. Note that all MEPDG performance model equations are provided in Appendix B.

MEPDG can provide fatigue damage predictions as the input values for the crack transfer function model and the distress predictions as the input values required by the IRI model. However, MEPDG does not output pavement response results which are key components for the rutting, faulting, fatigue, and thermal fracture models. Therefore, these prediction models could not be closed between inputs and outputs to be able to employ conventional optimization methodologies. These cases require numerous runs of MEPDG software to identify calibrated coefficients through a trial-and-error procedure.

A linear optimization approach using the sensitivity index was implemented as a screening procedure to reduce the computational burden of the trial-and-error procedure. In this linear optimization approach, the individual bias (ε_{ijk}) of each calibration coefficient per distress could be calculated by weight partition of total bias (ε_t) of all calibration coefficients per performance prediction determined from coefficient- normalized sensitivity index ($S^n{}_{ijk}$) as:

$$\varepsilon_{ijk} = \varepsilon_t \times S^n_{ijk} \tag{5}$$

Under the optimization constraint of $y_j^{measured} \simeq y_j^{\ local\text{-}predicted}$, the individual bias ($\varepsilon_{ijk}$) and the coefficient sensitivity index (S_{ijk}) could be expressed as:

$$\varepsilon_{ijk} = y_j^{measured} - y_j^{national-predicted} = y_j^{local-predicted} - y_j^{national-predicted} \tag{6}$$

$$S_{ijk} = \left.\frac{\Delta Y_j}{\Delta X_k}\right|_i = \frac{y_j^{local-predicted} - y_j^{national-predicted}}{x_k^{local} - x_k^{national}} = \frac{\varepsilon_{ijk}}{x_k^{local} - x_k^{National}} \tag{7}$$

$y_j^{measured}$ is the actual measured value for the performance prediction j; $y_j^{\ national\ -predicted}$ and $y_j^{\ local\ -predicted}$ are the values of the performance prediction j of nationally calibrated model coefficient, $x_k^{national}$ and locally calibrated model coefficient, x_k^{local}, respectively.

From equation (7), the locally calibrated model coefficient satisfying the optimization constraint could be derived as:

$$x_k^{local} = x_k^{National} + \frac{\varepsilon_{ijk}}{S_{ijk}} \tag{8}$$

The calculated locally calibrated model coefficient, x_k^{local}, is an approximate solution assuming linear relationship between the calibration coefficient and prediction. The trial-and-error procedure by running MEPDG based on the locally calibrated model coefficient, x_k^{local}, was found to more closely match the solution. This approach was also applied to identify the local calibration coefficients of the crack transfer function and IRI model when nonlinear programming optimization did not much improve the accuracy of performance predictions or provided underestimation of performance prediction. Note that overestimation of performance prediction can be considered a more conservative design approach when there is not much difference of bias compared to underestimation of performance predictions.

The MEDPG IRI prediction model consists of the primary distresses (e.g., total rutting, faulting) and a site factor along with calibration coefficients. The changes in distress predictions after local calibration of the associated distress models could result in the changes in IRI predictions even when using same nationally calibrated model coefficient of the IRI model. The predictions from: (1) the nationally calibrated IRI model inputs with nationally calibrated model coefficients, and (2) the locally calibrated model inputs with nationally calibrated model coefficients, were compared to the field measures values. If significant bias was identified from this comparison, the nationally calibrated model coefficient values of the IRI model were modified to reduce the bias of IRI model.

LOCAL CALIBRATION REUSLTS

The MEPDG was executed using the nationally calibrated model values to predict the performance indicators for each selected PMIS roadway section. The predicted performance measures were plotted relative to the measured values for the PMIS roadway sections. Based on the accuracy of performance predictions using the nationally calibrated model coefficient values, it was determined whether or not it was necessary to modify the national coefficient values for Iowa conditions. If needed, the locally calibrated model coefficients were identified to improve the accuracy of model predictions. The accuracy of performance predictions were evaluated by plotting the measurements against the predictions on a 45-degree line of equality, as well as by observing the average bias and standard error values. The average bias and standard error in this study are defined as

$$Ave\ Bias = \varepsilon_{ave} = \frac{\sum_{j=1}^{n}(y_j^{measured} - y_j^{predicted})}{n} \tag{9}$$

$$Stand.Error = \sqrt{\frac{\sum_{j=1}^{n}\left(y_j^{measured} - y_j^{predicted}\right)^2}{n}} \tag{10}$$

n is the number of data points in each distress comparison. The lower absolute value of average bias and standard error indicate better accuracy. A positive sign for the average bias indicates underestimated predictions. This process was applied to identify the calibration coefficients for Iowa JPCP and HMA performance prediction models as described below.

JPCP

The MEPDG new JPCP performance predictions include faulting, transverse cracking and IRI. Two models, namely the fatigue damage model and the transverse cracking transfer model, are involved in transverse cracking predictions. Fatigue model estimates fatigue damage and then transverse cracking transfer model converts fatigue damage estimation into transverse cracking predictions to equivalent transverse cracking measurements. Table 4 summarizes the nationally and locally calibrated model coefficients for JPCP performance predictions. The accuracy of each performance model with nationally and locally calibrated model coefficients are evaluated and discussed in the following subsection.

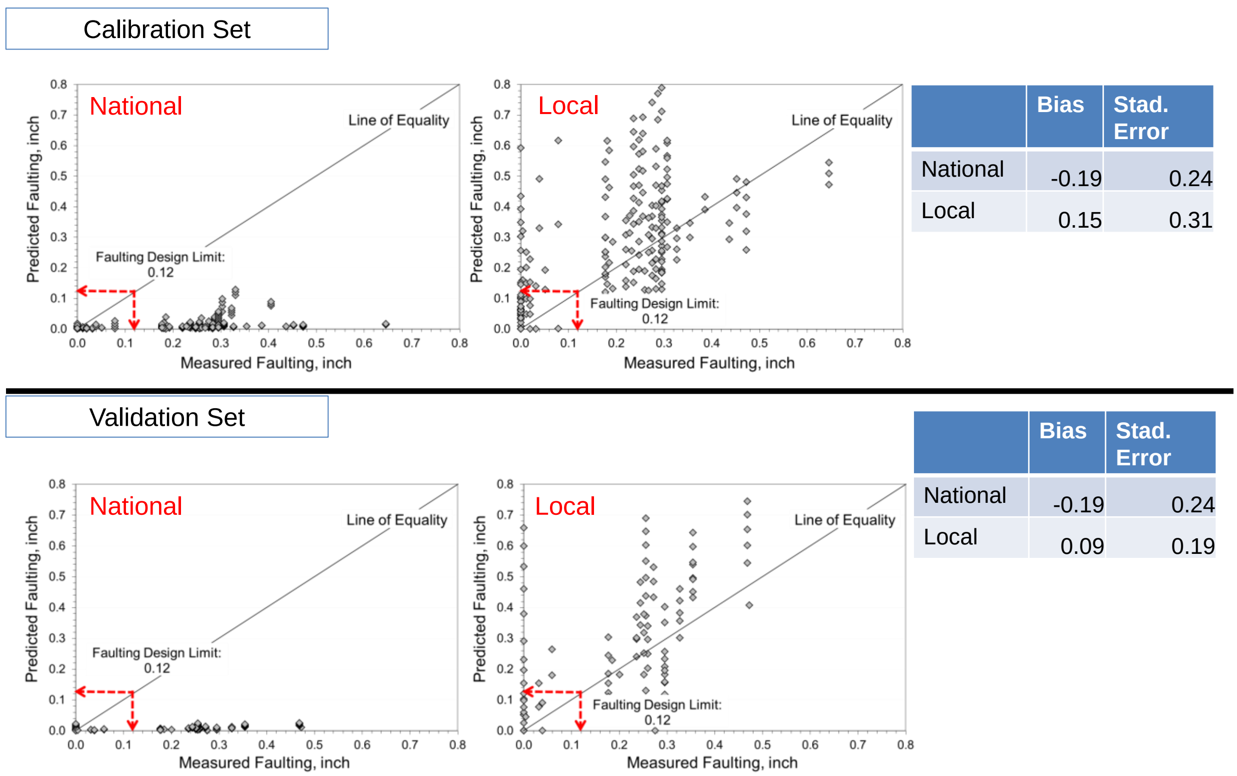
Table 4. Summary of calibration coefficients for JPCP performance predictions

Distress	Factors	National	Local
Faulting	C1	1.0184	2.0427
	C2	0.91656	1.83839
	C3	0.0021848	0.0043822
	C4	0.0008837	0.001772563
	C5	250	250
	C6	0.4	0.8
	C7	1.83312	1.83312
	C8	400	400
Fatigue for Crack	C1	2	2.17
	C2	1.22	1.32
Cracking	C4	1	1.08
	C5	-1.98	-1.81
IRI	C1	0.8203	0.04
	C2	0.4417	0.02
	C3	1.4929	0.07
	C4	25.24	1.17

Faulting

Figure 4 compares measured and predicted JPCP faulting predictions before and after local calibration for all sections utilized. As stated previously, about 70 % of the total selected sections were utilized to identify the local calibration factors while the remaining 30%, as an independent validation set, were utilized to verify the identified local calibration factors. The labels "Calibration Set" and "Validation Set" in Figure 4 denote comparisons between nationally calibrated and locally calibrated model predictions using the calibration and validation data sets, respectively.

The comparison suggests that the JPCP faulting model, after local calibration, yields more accurate predictions with respect to field measurements than the nationally-calibrated model, which severely under-predicts the extent of faulting. The positive sign of reduced bias values from the locally calibrated model predictions indicates lesser extent of overestimation. This change could make the design more conservative. The lower values of bias and standard error of locally calibrated model predictions from the validation data set suggest that the locally calibrated faulting model could improve the prediction accuracy even in other Iowa JPCP sections not used in the calibration procedures.



Calibration Set

	Bias	Stad. Error
National	-0.19	0.24
Local	0.15	0.31

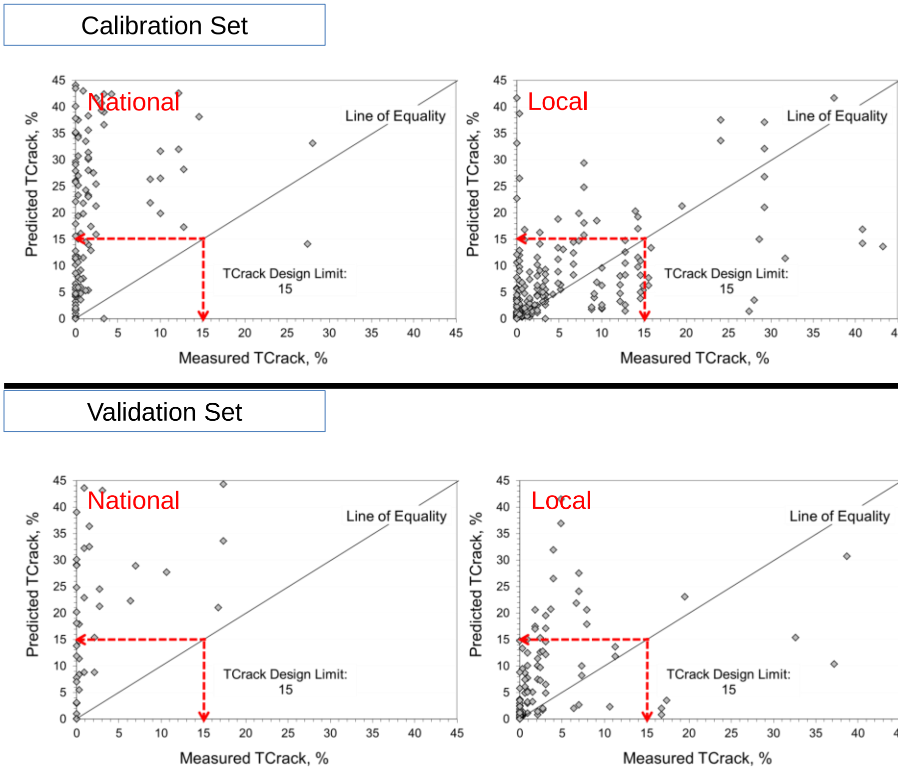
Validation Set

	Bias	Stad. Error
National	-0.19	0.24
Local	0.09	0.19

Figure 4. Overall summary of comparisons between measured and predicted JPCP faulting

Transverse Cracking

Figure 5 compares measured and predicted JPCP transverse cracking predictions before and after local calibration using the calibration and validation sets. The highly overestimated transverse cracking predictions using the nationally calibrated model coefficients moved more close to the line of equality when using the locally calibrated model coefficients. The lower values of bias and standard error also indicate that the transverse cracking prediction model was improved by modification of calibration coefficients for Iowa conditions.



Calibration Set

	Bias	Stad. Error
National	36.37	46.83
Local	3.38	20.88

Validation Set

	Bias	Stad. Error
National	41.37	51.09
Local	5.17	12.94

Figure 5. Overall summary of comparisons between measured and predicted JPCP transverse cracking

IRI

The local calibration of IRI model for JPCP involved the calibration of distress models (faulting and transverse cracking) as IRI model inputs and the calibration of associated coefficients to each distress input in the IRI model. Figure 6 compares the measured and predicted JPCP IRI predictions before and after local calibration using the calibration and validation sets. The nationally calibrated IRI model predictions overestimated the measured values while the locally calibrated IRI model predictions were placed on the line of equality. The lower values of bias and standard error also indicate that the locally calibrated IRI model provide better estimation of the measured values.

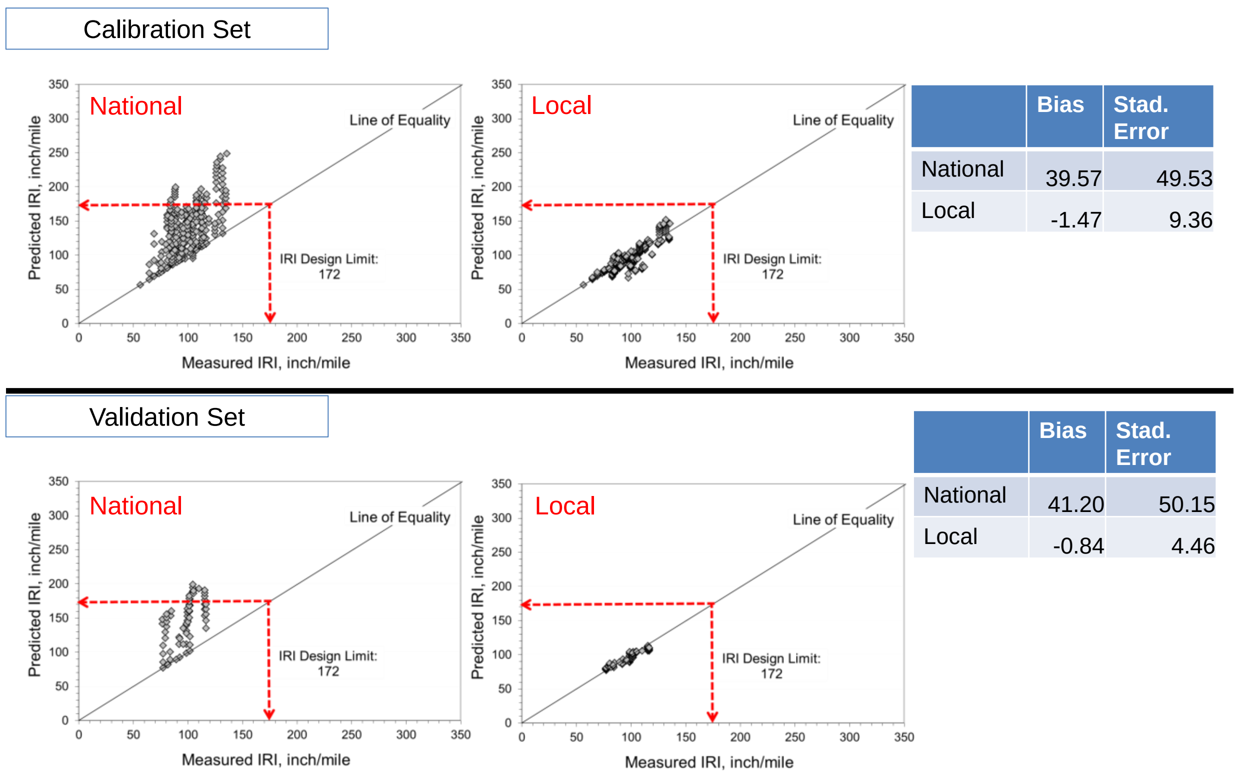


Figure 6. Overall summary of comparisons between measured and predicted JPCP IRI

HMA Pavement

The MEPDG new HMA pavement performance predictions include rutting, longitudinal (top down) cracking, alligator cracking (bottom up) cracking, thermal (transverse) cracking and IRI. Rutting predictions consist of HMA layer rutting, granular base rutting, subgrade rutting and total surface rutting. Similar to JPCP, the HMA fatigue models were utilized to estimate fatigue damage, which were input to the transfer function models of longitudinal cracking and alligator cracking and converted to equivalent cracking measurements.

Table 5 summarizes the nationally and locally calibrated model coefficients for new HMA pavement performance predictions. The accuracy of each performance models with the nationally and locally calibrated model coefficients are evaluated and discussed in the following subsection.

Table 5. Summary of calibration coefficients for HMA performance predictions

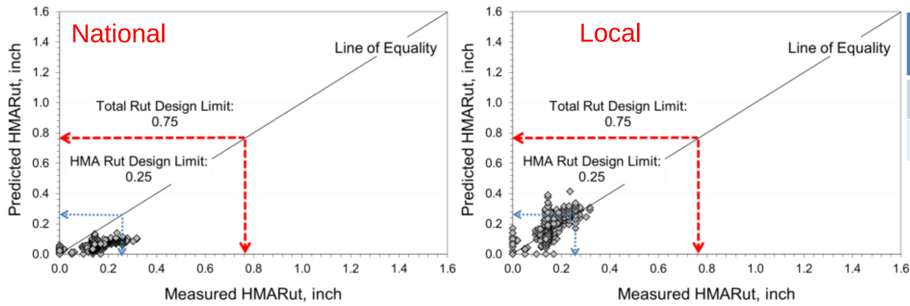
Distress	Factors	National	Local
HMA Rut	B1	1	1
	B2	1	1.15
	B3	1	1
GB Rut	B1_Granular	1	0*
SG Rut	B1_Fine-grain	1	0*
Fatigue for ACrack and LCrack	B1	1	1
	B2	1	1
	B3	1	1
LCrack	C1_Top	7	0.82
	C2_Top	3.5	1.18
	C4_Top	1,000	1,000
ACrack	C1_Bottom	1	1
	C2_Bottom	1	1
	C4_Bottom	6,000	6,000
TCrack	K_Level 3	1.5	1.5
IRI	C1	40	40
	C2	0.4	0.4
	C3	0.008	0.008
	C4	0.015	0.015

* Minimum acceptable value (0.001) in use of DARWin-ME^{TM}

Rutting

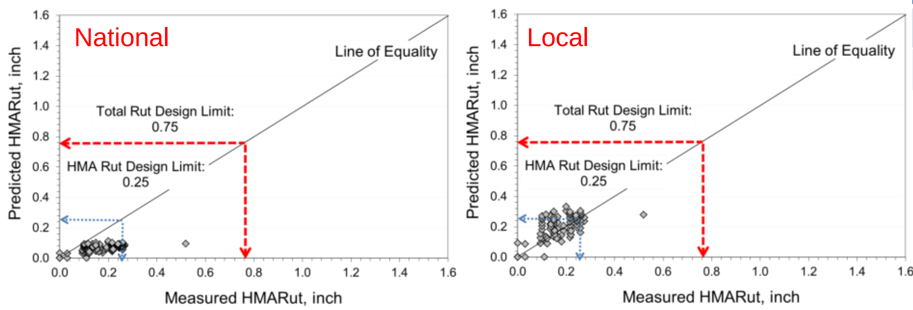
The comparison between measured and predicted rutting before and after local calibration for all sections utilized are presented in Figure 7 to Figure 10. Each of these figures compares rutting measurements and predictions for each pavement layer. Figure 7 demonstrates that the HMA layer rutting predictions after local calibration were placed more close to the line of equality. The lower values of bias and standard error also indicate that the locally calibrated HMA rutting model provide better predictions of HMA layer rutting measurements. Figure 8 and Figure 9 illustrate that the calibrated rutting model predictions for granular and subgrade layer are more close to rutting measurements in Iowa HMA pavement systems. Figure 10 presents the comparisons for accumulated (total) rutting. The lower values of bias and stander error indicate that the locally calibrated HMA rutting prediction model could improve the accuracy of accumulated rutting predictions for Iowa HMA pavements.

Calibration Set



	Bias	Stad. Error
National	-0.08	0.10
Local	0.03	0.07

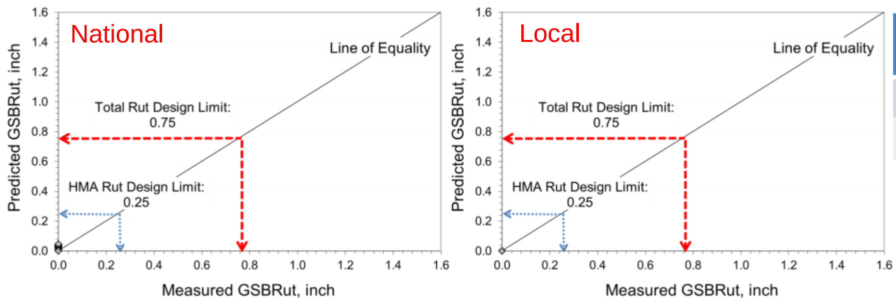
Validation Set



	Bias	Stad. Error
National	-0.10	0.12
Local	0.02	0.07

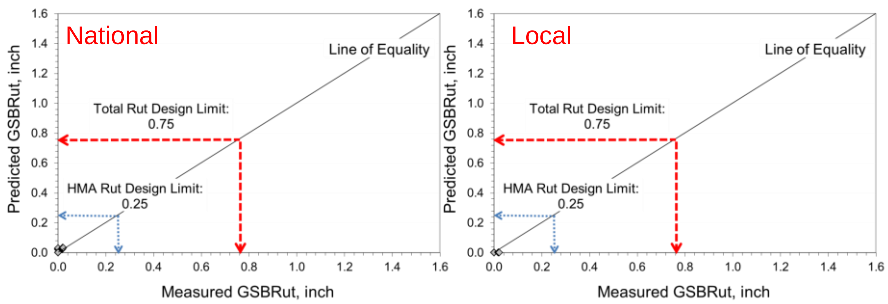
Figure 7. Overall summary of comparisons between measured and predicted HMA layer rutting for HMA pavements

Calibration Set



	Bias	Stad. Error
National	0.00	0.01
Local	0.00	0.00

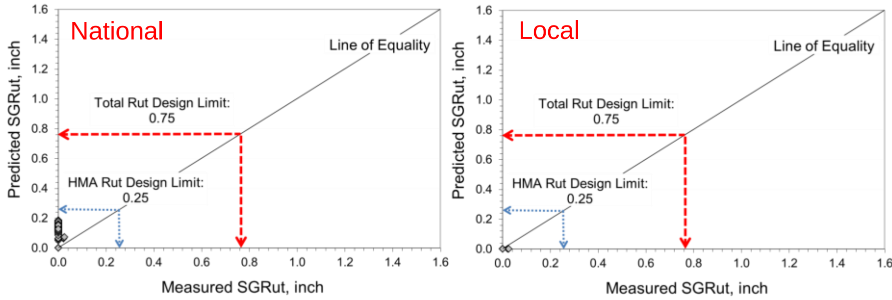
Validation Set



	Bias	Stad. Error
National	0.00	0.01
Local	0.00	0.01

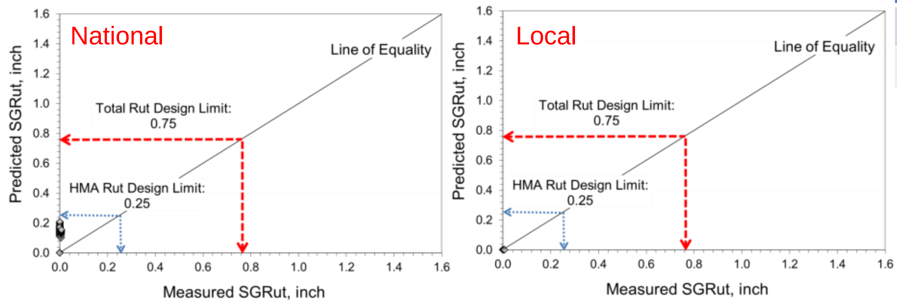
Figure 8. Overall summary of comparisons between measured and predicted granular base layer rutting for HMA pavements

Calibration Set



	Bias	Stad. Error
National	0.13	0.14
Local	0.00	0.00

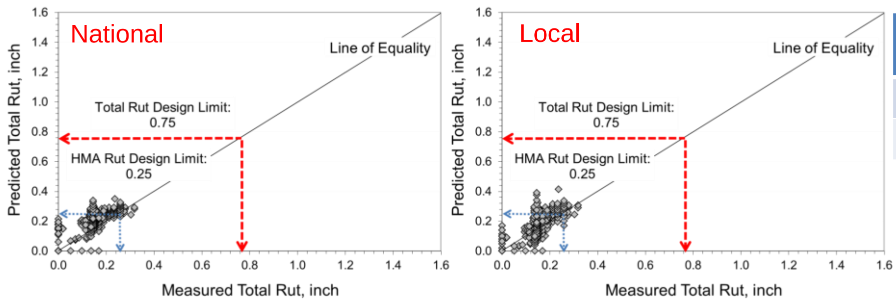
Validation Set



	Bias	Stad. Error
National	0.13	0.14
Local	0.00	0.00

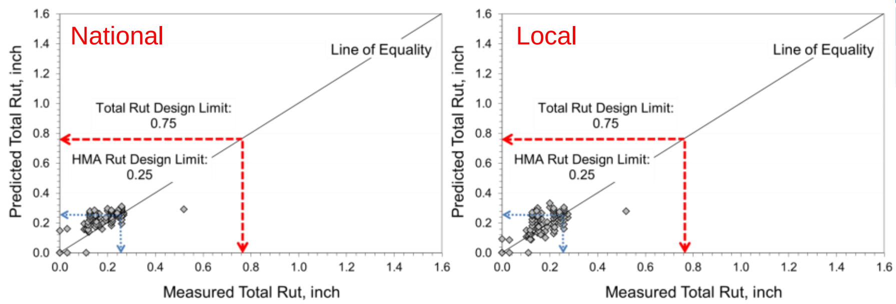
Figure 9. Overall summary of comparisons between measured and predicted subgrade layer rutting for HMA pavements

Calibration Set



	Bias	Stad. Error
National	0.05	0.08
Local	0.03	0.07

Validation Set

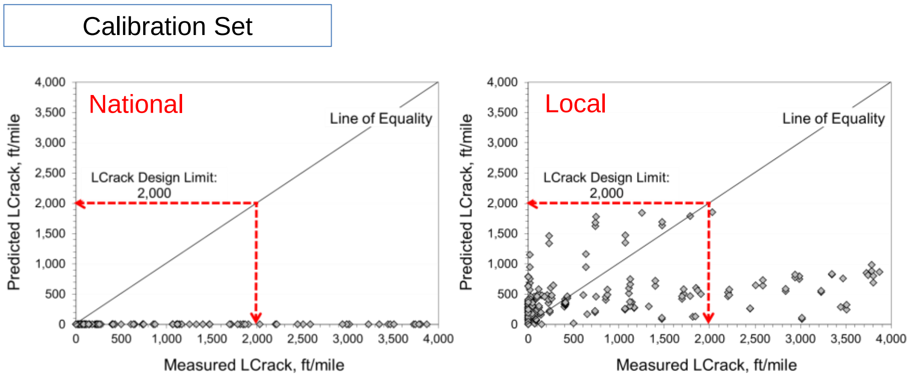


	Bias	Stad. Error
National	0.04	0.07
Local	0.02	0.07

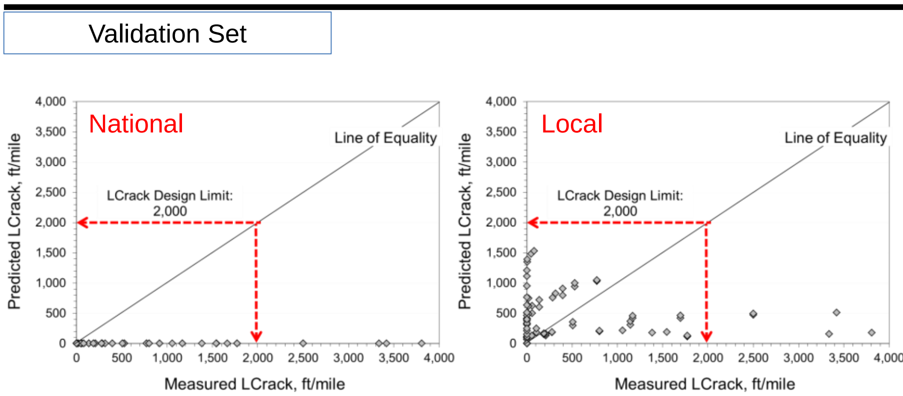
Figure 10. Overall summary of comparisons between measured and predicted total rutting for HMA pavements

Longitudinal (Top-Down) Cracking

Figure 11 demonstrates that the locally calibrated longitudinal cracking model gives better predictions with lower bias and standard errors while the nationally calibrated model severely under-predicts the extent of longitudinal cracking. Improved HMA longitudinal cracking prediction models are currently being developed under NCHRP projects (Roque et al 2010, NCHRP 2012).



	Bias	Stad. Error
National	-1,316	3,039
Local	-909	2,767



	Bias	Stad. Error
National	-1,332	3,232
Local	-872	2,958

Figure 11. Overall summary of comparisons between measured and predicted longitudinal cracking for HMA pavements

Alligator (Bottom-Up) Cracking

Figure 12 compares the HMA alligator cracking measurements to corresponding predictions obtained using the nationally calibrated alligator cracking prediction model. The predictions provide good estimation to the measurements with lower bias and standard error. Only two data points among a total of 327 data sets show underestimation of predictions to measurements but are still placed within the design limit of 25%. Thus, the nationally calibrated alligator cracking model did not require local calibration for Iowa conditions at this stage.

	Bias	Stad. Error
National	-0.14	1.22

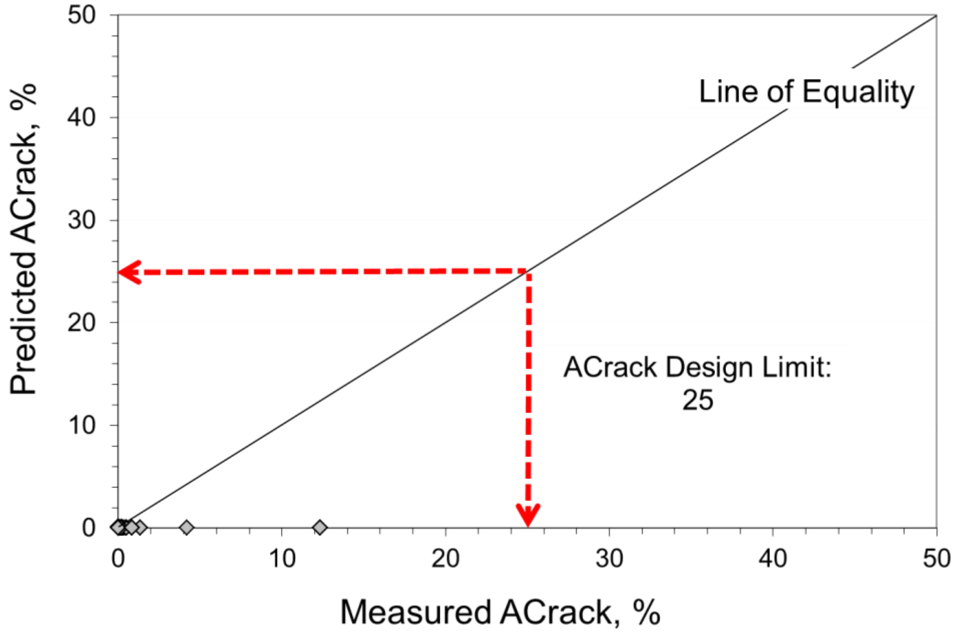


Figure 12. Overall summary of comparisons between measured and predicted alligator cracking for HMA pavements

Thermal (Transverse) Cracking

Previous studies reported that little or no thermal cracking was predicted when using the proper binder grade for local climate conditions (Hall et al. 2011, Schwartz et al. 2012). As seen in Figure 13, minimal predictions from nationally calibrated thermal cracking model are observed while significant thermal cracking measurements are actually observed in the field. In addition, the modification of calibration coefficients in the MEPDG thermal cracking model could not provide changes in predictions. Therefore, the HMA thermal cracking model was not considered for local calibration in this study. Improved thermal cracking models have been developed under FHWA pooled fund studies (TPF 2012, Marasteanu et al. 2012).

	Bias	Stad. Error
National	-446	1,203

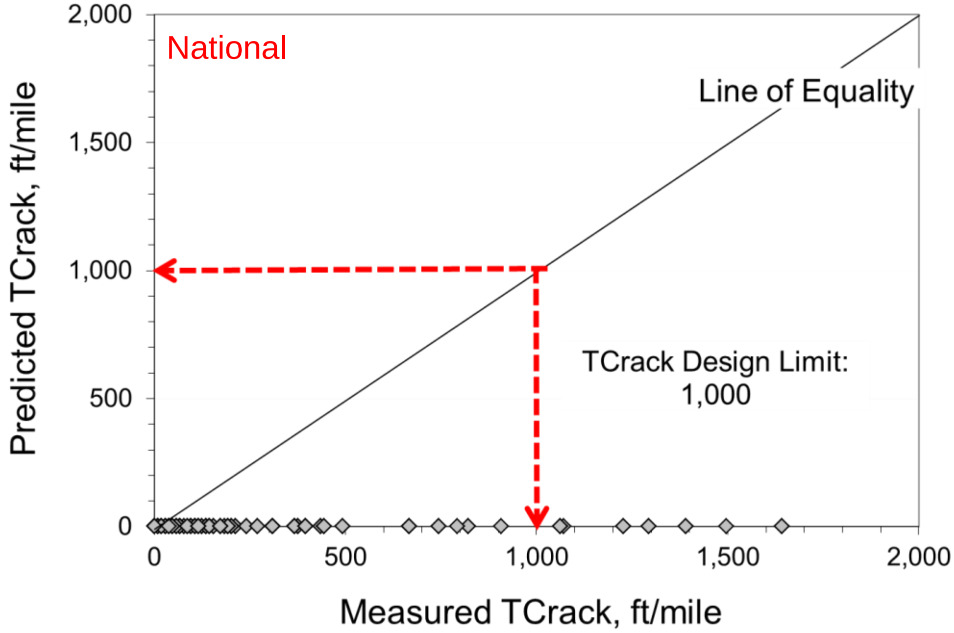
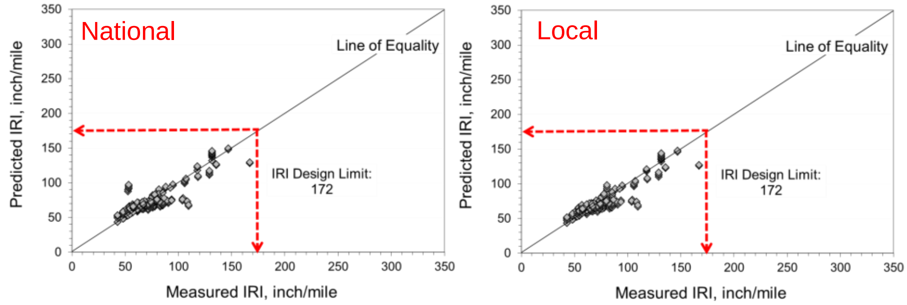


Figure 13. Overall summary of comparisons between measured and predicted transverse cracking for HMA pavements

IRI

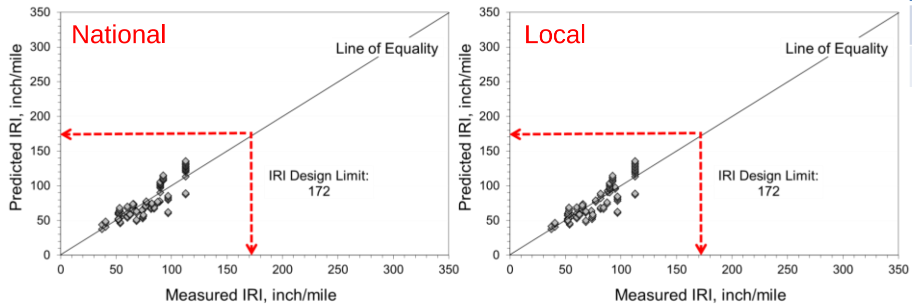
Figure 14 compare the measured IRI values with predictions from (1) IRI model containing nationally calibrated distress model inputs with nationally calibrated model coefficients and (2) IRI model containing locally calibrated distress model inputs with nationally calibrated model coefficients. Both IRI models provide good estimation to field measurements. Further modification to nationally calibrated IRI model coefficients was not considered because (1) good estimation of IRI measurements could be obtained without modification of calibration coefficients and (2) the examination and improvement of HMA longitudinal cracking and thermal cracking models are being carried out through national studies.

Calibration Set



	Bias	Stad. Error
National	-0.67	12.35
Local	-2.19	10.79

Validation Set



	Bias	Stad. Error
National	-0.73	13.23
Local	-0.70	12.83

Figure 14. Overall summary of comparisons between measured and predicted IRI for HMA pavements

HMA over JPCP

The MEPDG HMA over JPCP performance predictions include rutting, longitudinal (top-down) cracking, alligator cracking (bottom-up) cracking, thermal (transverse) cracking, reflection cracking, and IRI. Rutting predictions consist of HMA layer rutting, granular base layer rutting, subgrade layer rutting and total pavement rutting. However, most of the total rutting predictions are assumed to come from HMA layer because the existing JPCP can provide strong foundation to HMA surface overlay to prevent granular base and subgrade layer rutting. Similar to previous pavement types, the fatigue models were utilized to estimate fatigue damage which were inputted into transfer functions of longitudinal cracking and alligator cracking predictions to obtain equivalent cracking measurements.

Table 6 summarizes the nationally and locally calibrated model coefficients for HMA over JPCP performance predictions. The accuracy of each performance model using the nationally and locally calibrated model coefficients are evaluated and discussed in the following subsection.

Table 6. Summary of calibration coefficients for HMA over JPCP performance predictions

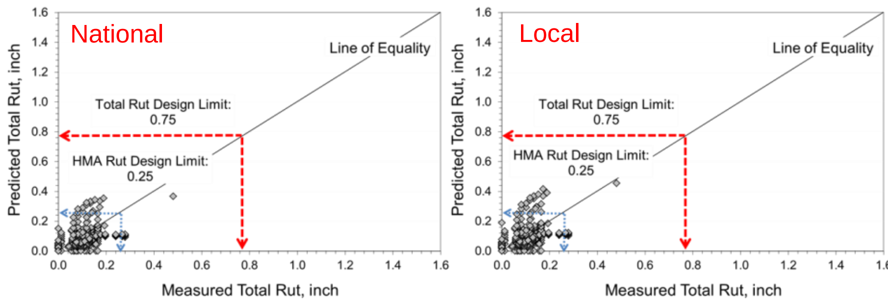
Distress	Factors	National	Local
HMA Rut	B1	1	1
	B2	1	1.007
	B3	1	1.007
GB Rut	B1_Granular	1	0*
SG Rut	B1_Fine-grain	1	0*
Fatigue for ACrack and LCrack	B1	1	1.00
	B2	1	0.800
	B3	1	0.800
LCrack	C1_Top	7	7
	C2_Top	3.5	3.5
	C4_Top	1,000	1,000
ACrack	C1_Bottom	1	1
	C2_Bottom	1	1
	C4_Bottom	6,000	6,000
TCrack	K_Level 3	1.5	1.5
RCrack	C	1	1
	D	1	1
IRI	C1	40.8	40.8
	C2	0.575	0.575
	C3	0.0014	0.0014
	C4	0.00825	0.00825

* Minimum acceptable value (0.001) in use of DARWin-ME™

Rutting

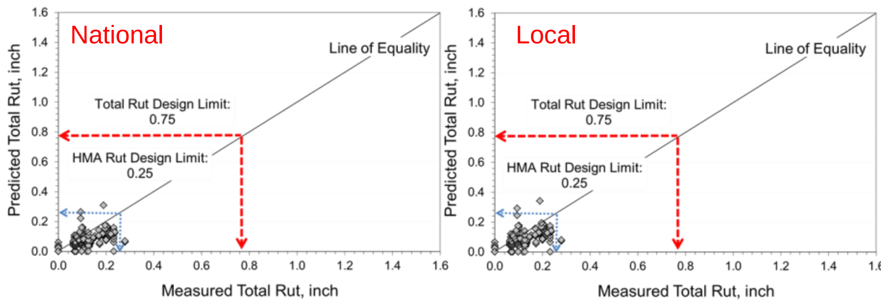
The comparisons between measured and predicted rutting before and after local calibration using the calibration and validation sections are presented in Figure 15. Both the nationally and locally calibrated rutting models provide good estimation to field measurements. After local calibration, the accuracy of rutting predictions improved little, but this improvement is not considered significant.

Calibration Set



	Bias	Stad. Error
National	-0.017	0.069
Local	-0.009	0.072

Validation Set

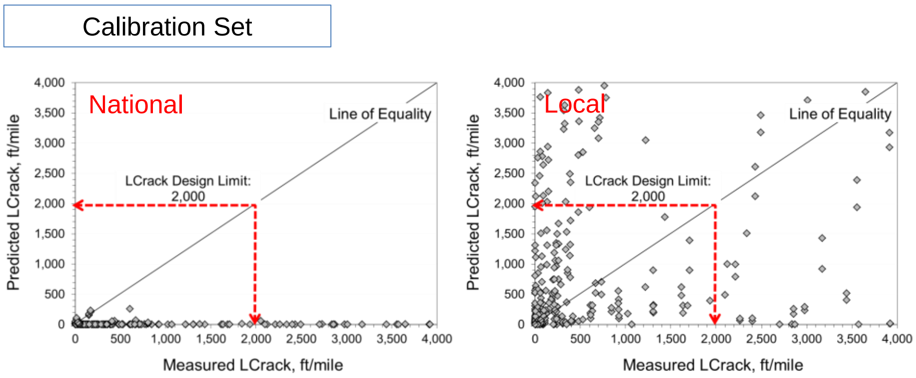


	Bias	Stad. Error
National	-0.041	0.077
Local	-0.034	0.075

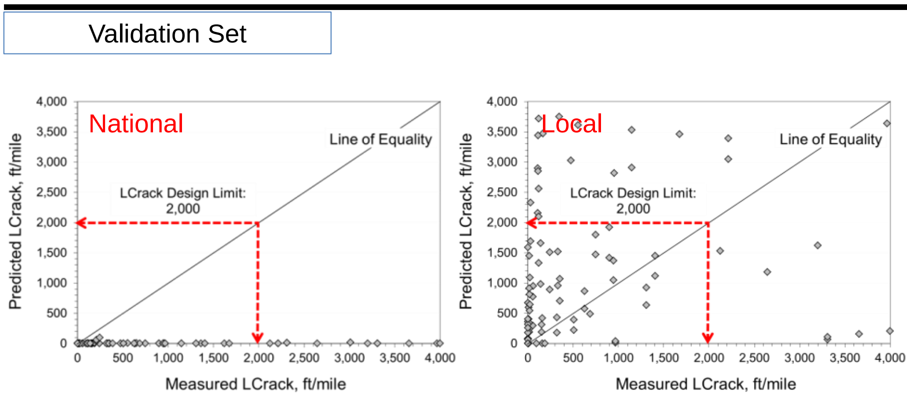
Figure 15. Overall summary of comparisons between measured and predicted total rutting for HMA over JPCP

Longitudinal (Top-Down) Cracking

As seen in Figure 16, the locally calibrated longitudinal cracking models produce better predictions with lower bias and standard errors than the nationally calibrated model predictions. However, the modification of longitudinal cracking related model coefficients in the MEPDG could not tighten the scatter (predictions vs. measurements) around the line of equality. As mentioned previously, improvements and further refinements to the HMA longitudinal cracking model are being made through national studies (Roque et al. 2010, NCHRP 2012).



	Bias	Stad. Error
National	-1,212	2,358
Local	636	3,219

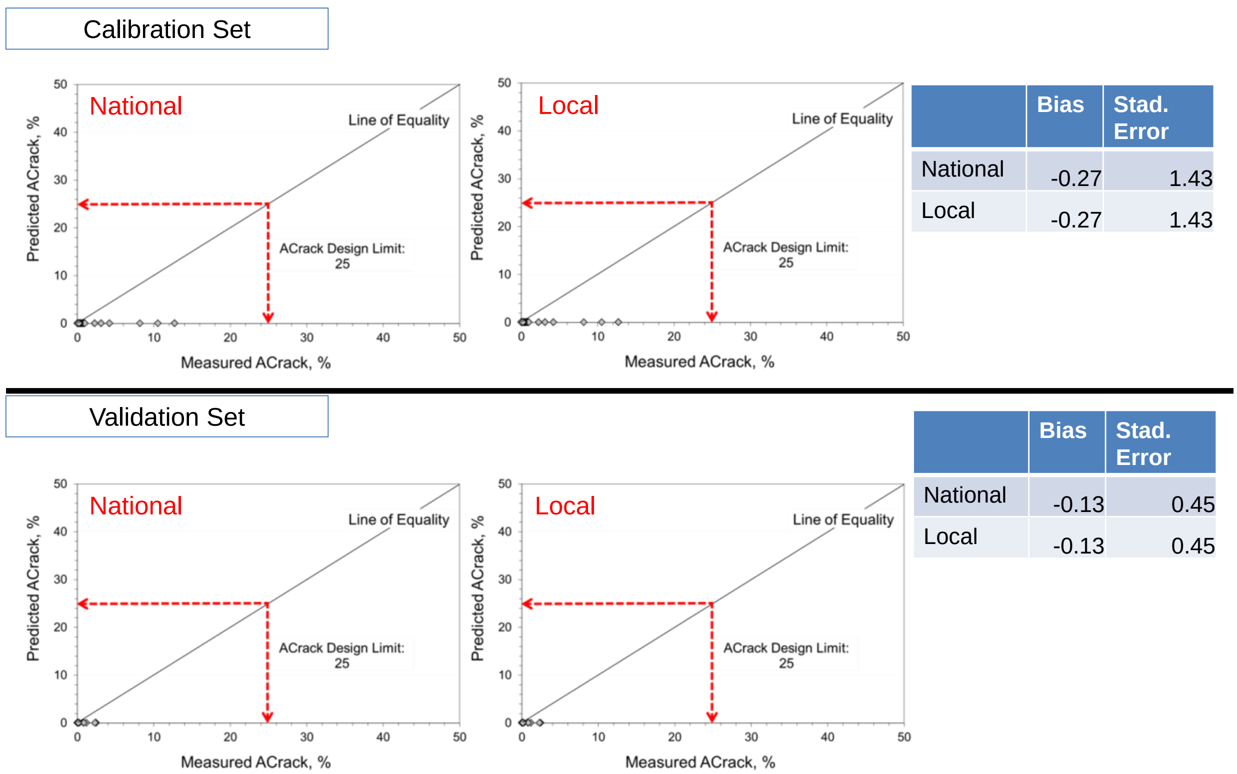


	Bias	Stad. Error
National	-842	1,659
Local	988	2,327

Figure 16. Overall summary of comparisons between measured and predicted longitudinal cracking for HMA over JPCP

Alligator (Bottom-Up) Cracking

Figure 17 presents the comparison between measured and predicted alligator cracking. The nationally calibrated model predictions provide good estimation to the measurements with lower bias and standard error. Only five among a total of 490 data sets show underestimation of predictions, but are still placed within the design limit of 25%. The fatigue damage model coefficients were modified mainly for improvement of longitudinal cracking predictions. The modification of fatigue damage model coefficients did not reduce the accuracy of alligator cracking predictions (See locally calibrated model predictions in Figure 17). Note that the alligator cracking predictions are estimated from fatigue damage and alligator cracking transfer function models.



	Bias	Stad. Error
National	-0.27	1.43
Local	-0.27	1.43

	Bias	Stad. Error
National	-0.13	0.45
Local	-0.13	0.45

Figure 17. Overall summary of comparisons between measured and predicted alligator cracking for HMA over JPCP

Thermal (Transverse) Cracking and Reflection Cracking

Figure 18 demonstrates poor prediction accuracy of nationally calibrated thermal cracking model for Iowa HMA over JPCP. As discussed in the previous section, thermal cracking models are still evolving and M-E based reflection cracking model from the recently completed NCHRP project 1-41 (Lytton et al. 2010) is not yet adapted in the MEPDG and DARWin-ME™. In addition to this, transverse cracking measurements recorded in Iowa DOT PMIS do not differentiate thermal cracking and reflection cracking measurements. Thus, thermal cracking and reflection cracking were not considered in this study.

	Bias	Stad. Error
National	-1,963	3,110

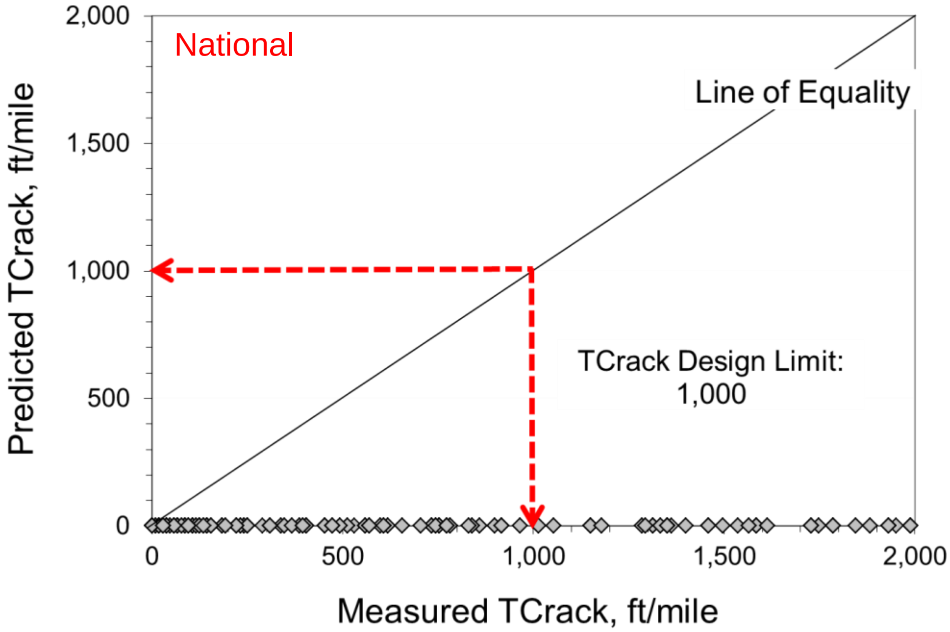
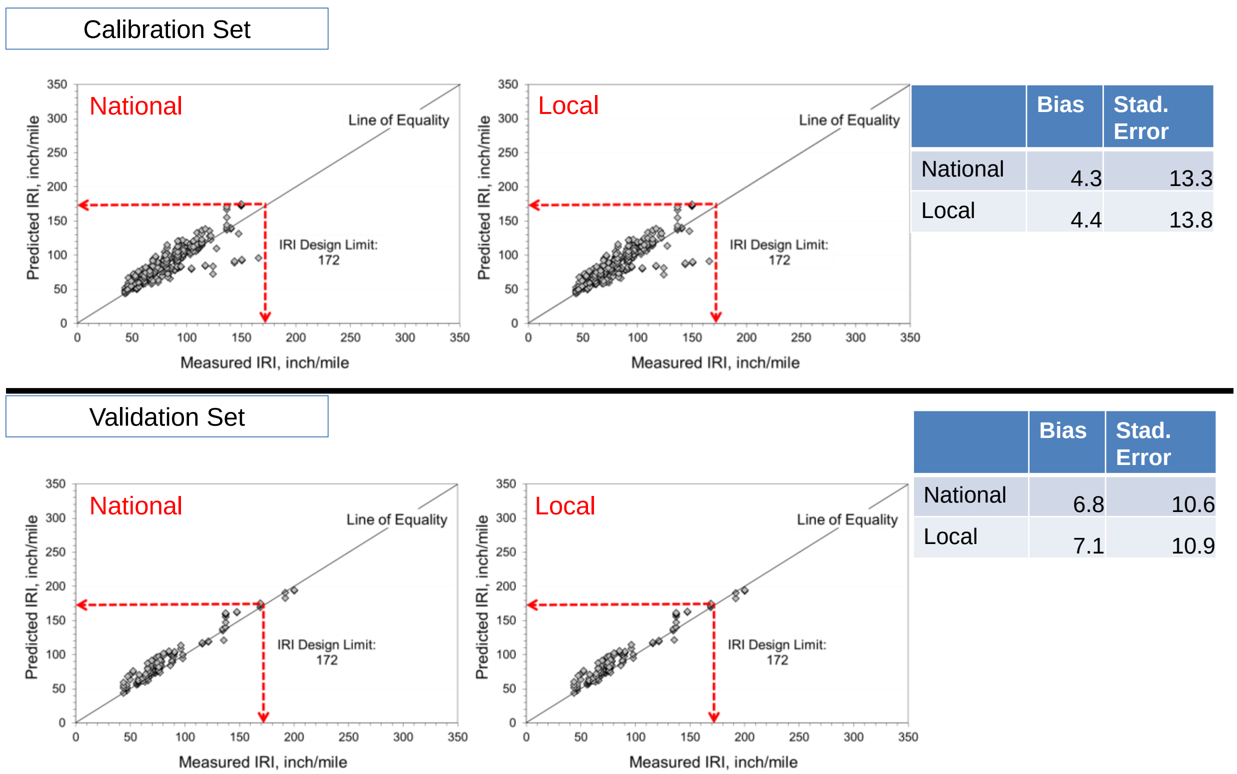


Figure 18. Overall summary of comparisons between measured and predicted thermal cracking for HMA over JPCP

IRI

Figure 19 compares the measured IRI values with predictions from (1) IRI model containing nationally calibrated distress model inputs with nationally calibrated model coefficients and (2) IRI model containing locally calibrated distress model inputs with nationally calibrated model coefficients. Both IRI models provide good estimation to field measurements. Further modification to nationally calibrated IRI model coefficients were not considered because good estimation to IRI could be obtained without modification of calibration coefficients. In addition, the examination and improvement of some distress prediction models (such as longitudinal cracking and thermal cracking) are currently being carried out through national studies.



Calibration Set

	Bias	Stad. Error
National	4.3	13.3
Local	4.4	13.8

Validation Set

	Bias	Stad. Error
National	6.8	10.6
Local	7.1	10.9

Figure 19. Overall summary of comparisons between measured and predicted IRI for HMA over JPCP

COMPARISON BETWEEN MEPDG AND DARWIN-ME™ PREDICTIONS: PRELIMINARY STUDY

The DARWin-ME™ released in April 2011 builds upon the latest version of research grade MEPDG software (version 1.1). Key features and enhancements in DARWin-ME™ over the MEPDG are found in DARWin-ME™ help manual (AASHTO 2011). The DARWin-ME™ has been renamed as Pavement ME Design recently. A preliminary study was undertaken to compare MEPDG (version 1.1) performance predictions with those of DARWin-ME™ (version 1.1.32) for JPCP, HMA pavement and HMA over JPCP to observe if there are any differences in trends and magnitudes of performance predictions outputted by both software.

The modeled JPCP section consisted of an 8-in thick PCC slab with 20-ft transverse joint spacing over a 6-in cement treated base (CTB), a 6-in crushed granular subbase, and an A-7-6 compacted embankment subgrade. The modeled HMA pavement section consisted of an 8-in thick HMA (PG 58-28 binder grade) surface over a 4-in HMA (PG 58-28 binder grade) base, and an A-6 compacted embankment subgrade. The modeled HMA over JPCP section consisted of a 5-in thick HMA (PG 58-28 binder grade) surface over a 10-in exiting PCC slab with 20-ft transverse joint spacing, a 5-in crushed granular subbase, and an A-6 compacted embankment subgrade.

A 30-year design life for JPCP, a 20-year design life for HMA and a 20-year design life for

HMA over JPCP with two reliability levels (50% and 90%) were utilized. Two levels of AADTT were utilized (1,000 and 5,000) and two climate site locations (Des Moines, Iowa and International Falls airport, Minnesota) were considered. The default national/global calibration coefficients were utilized in making pavement performance predictions.

The DARWin-ME™ software allows the user to import climate data in XML format generated in DARWin-ME™ as well as the ICM format climate data file generated in MEPDG. However, DARWin-ME™ requires more hourly climate data points rather than MEPDG. Error or warning messages were displayed in the error list pane area of the program when ICM format climate data files generated from MEPDG for both climate site locations were imported into DARWin-ME™. Thus, it was not possible to use the same format of climate file in both DARWin-ME™ and MEPDG during this study. In this comparison, the DARWin-ME™ utilized the XML climate file format and the MEPDG utilized the ICM climate data format for the same climatic locations (Des Moines, Iowa and International Falls airport, Minnesota). Except for the climate file format, all design input values required in both DARWin-ME™ and MEPDG were identical.

Table 7 summarizes JPCP design life performance prediction comparisons between MEPDG and DARWin-ME™ with nationally calibrated performance prediction models. For added insight, those performance measures and their magnitudes that are dissimilar between MEPDG and DARWin-ME™ are indicated by the shaded cells in the table. JPCP faulting and transverse cracking predictions from MEPDG and DARWin-ME™ do not show significant differences at both climate site locations. However, the national IRI predictions from MEPDG and DARWin-ME™ have differences at both climate site locations. The IRI model in both MEPDG and DARWin-ME™ is an empirical relation consisting of transverse cracking, the joint faulting and site specifics. Since transverse cracking and the joint faulting predictions in both MEPDG and DARWin-ME™ are similar, it is suspected that the national IRI prediction differences between MEPDG and DARWin-ME™ might have arisen from site specifics having climate related variables (freezing index and number of freezing cycles). Note that the XML climate file in DARWin-ME™ has more hourly climate data points than ICM climate data format in MEPDG. However, further research is warranted to investigate these differences. Note that only two climatic conditions were evaluated in this preliminary study.

Table 8 summarizes HMA design life performance prediction comparison results between MEPDG and DARWin-ME™ with nationally calibrated performance prediction models. For HMA pavement, longitudinal cracking and transverse cracking predictions show some differences between MEPDG and DARWin-ME™ at both climate site locations. Similar to JPCP, the IRI predictions for the International Falls airport are different for MEPDG and DARWin-ME™.

Table 9 summarizes the HMA over JPCP design life performance prediction comparison results between MEPDG and DARWin-ME™ using the nationally calibrated performance prediction models. Transverse cracking and IRI predictions show some differences between MEPDG and DARWin-ME™ at both climate site locations.

The results from this preliminary study indicated that the differences between the predictions of the two software versions are quite significant, at least in some cases, warranting further investigation to determine if the local calibration study needs to be repeated using the latest version of the DARWin-ME™ solution which is now renamed as Pavement ME Design (version 1.3).

Table 7. JPCP performance prediction comparisons between MEPDG and DARWin-ME™ using nationally calibrated performance prediction models

AADTT	Reliability (%)	Distress	Des Moines, IA		Falls Airport, MN	
			MEPDG 1.1	DARWin-ME	MEPDG 1.1	DARWin-ME
1,000	50	***IRI (in/mi)***	***93.8***	***67.4***	***152.0***	***70.8***
		TCracking (% slabs)	0.0	0.0	0.1	0.1
		Faulting (in)	0.001	0.001	0.003	0.003
1,000	90	***IRI (in/mi)***	***132.5***	***94.4***	***208.2***	***100.1***
		TCracking (% slabs)	4.5	4.5	4.9	4.8
		Faulting (in)	0.021	0.020	0.026	0.026
5,000	50	***IRI (in/mi)***	***97.2***	***70.8***	***159.0***	***78.2***
		TCracking (% slabs)	0.7	0.7	2.0	1.8
		Faulting (in)	0.006	0.006	0.012	0.013
5,000	90	***IRI (in/mi)***	***138.2***	***101.0***	***217.9***	***112.8***
		TCracking (% slabs)	6.6	6.6	9.1	8.7
		Faulting (in)	0.032	0.030	0.042	0.044

Note: The shaded cells highlight dissimilarities between MEPDG and DARWin-ME™ predictions

Table 8. HMA prediction comparisons between MEPDG and DARWin-ME™ using nationally calibrated performance prediction models

AADTT	Reliability (%)	Distress	Des Moines, IA		Falls Airport, MN	
			MEPDG 1.1	DARWin-ME	MEPDG 1.1	DARWin-ME
1,000	50	***IRI (in/mi)***	111.8	111.9	***111.8***	***124.5***
		LCracking (ft/mile)	***0.4***	***1.2***	***0.1***	***0.6***
		ACracking (%)	0.2	0.2	0.1	0.1
		TCrack (ft/mi)	***1.0***	***3.4***	***1.0***	***1603.2***
		Rutting (Total) (in)	0.4	0.4	0.4	0.4
1,000	90	***IRI (in/mi)***	151.5	151.5	***151.5***	***168.0***
		LCracking (ft/mile)	***394.0***	***311.2***	***328.7***	***274.4***
		ACracking (%)	1.6	1.6	1.6	1.6
		TCrack (ft/mi)	***84.3***	***31.3***	***84.3***	***2445.5***
		Rutting (Total) (in)	0.5	0.5	0.4	0.5
5,000	50	***IRI (in/mi)***	121.5	121.5	***120.2***	***132.8***
		LCracking (ft/mile)	***4.8***	***14.3***	***1.6***	***6.5***
		ACracking (%)	0.9	0.8	0.7	0.6
		TCrack (ft/mi)	***1.0***	***3.4***	***1.0***	***1603.2***
		Rutting (Total) (in)	0.6	0.6	0.6	0.6
5,000	90	***IRI (in/mi)***	163.7	163.7	***162.0***	***178.4***
		LCracking (ft/mile)	***796.0***	***1380.9***	***560.0***	***741.5***
		ACracking (%)	2.3	2.3	2.1	2.1
		TCrack (ft/mi)	***84.3***	***31.3***	***84.3***	***2445.5***
		Rutting (Total) (in)	0.8	0.8	0.7	0.7

Note: The shaded cells highlight dissimilarities between MEPDG and DARWin-ME™ predictions

Table 9. HMA over JPCP prediction comparisons between MEPDG and DARWin-ME™ using nationally calibrated performance prediction models

AADTT	Reliability (%)	Distress	Des Moines, IA		Falls Airport, MN	
			MEPDG 1.1	DARWin-ME	MEPDG 1.1	DARWin-ME
1,000	50	***IRI (in/mi)***	***113.3***	***88.4***	***108.4***	***90.6***
		LCracking (ft/mile)	7.5	7.4	7.3	7.4
		ACracking (%)	0.0	0.0	0.0	0.0
		RCracking (%)	7.8	7.7	7.8	7.7
		TCrack (ft/mi)	***1.0***	***128.0***	***1.0***	***2226.8***
		Rutting (Total) (in)	0.2	0.2	0.2	0.1
1,000	90	***IRI (in/mi)***	***153.5***	***120.9***	***147.2***	***123.9***
		LCracking (ft/mile)	932.7	826.4	922.6	827.2
		ACracking (%)	1.5	1.5	1.5	1.5
		TCrack (ft/mi)	***84.3***	***219.3***	***84.3***	***3386.4***
		Rutting (Total) (in)	0.2	0.2	0.2	0.2
5,000	50	***IRI (in/mi)***	***121.0***	***96.3***	***118.9***	***97.1***
		LCracking (ft/mile)	86.0	84.9	83.4	85.1
		ACracking (%)	0.0	0.0	0.0	0.0
		RCracking (%)	7.8	7.7	7.8	7.7
		TCrack (ft/mi)	***1.0***	***128.0***	***1.0***	***2226.8***
		Rutting (Total) (in)	0.4	0.4	0.3	0.3
5,000	90	***IRI (in/mi)***	***163.3***	***131.4***	***160.7***	***132.5***
		LCracking (ft/mile)	2027.4	2949.8	2010.3	2951.6
		ACracking (%)	1.5	1.5	1.5	1.5
		TCrack (ft/mi)	***84.3***	***219.3***	***84.3***	***3386.4***
		Rutting (Total) (in)	0.5	0.5	0.4	0.4

Note: The shaded cells highlight dissimilarities between MEPDG and DARWin-ME™ predictions

SUMMARY

This research aims to improve the accuracy of MEPDG projected pavement performance predictions for Iowa pavement systems through local calibration of MEPDG performance prediction models. A total of 35 JPCP sections representing rigid pavements, a total of 35 HMA sections representing flexible pavements, and 60 HMA over JPCP sections representing composite pavements were selected. The required MEPDG inputs for the selected sections were collected primarily from Iowa DOT PMIS, material testing records and previous project reports relevant to MEPDG implementation in Iowa. A database of historical performance data for the selected sections was prepared from the Iowa DOT PMIS. The accuracy of the nationally calibrated MEPDG prediction models for Iowa conditions was evaluated. The local calibration factors of MEPDG prediction models were identified using both linear and nonlinear optimization approaches to improve the accuracy of model predictions.

The local calibration coefficients identified through this study are summarized and presented in Table 4 for JPCP, Table 5 for HMA, and Table 6 for HMA over JPCP. Based on this study, the following conclusions were made for each pavement type and the corresponding performance prediction models. Recommendations on the use of identified local calibration coefficients as well as future research are also provided.

Conclusions: JPCP

- The locally calibrated faulting model for Iowa JPCP gives better predictions with lower bias and standard errors than the nationally calibrated model with severely underestimated faulting measures.
- The locally calibrated transverse cracking model for Iowa JPCP gives better predictions with lower bias and standard errors than the nationally calibrated model with severely overestimated transverse cracking measures.
- The locally calibrated IRI model for Iowa JPCP improves the accuracy of predictions by tightening the scatter around the line of equality. The nationally calibrated model overestimates IRI measures.

Conclusions: HMA Pavement

- Both nationally as well as locally calibrated rutting models provide good predictions of the total (accumulated) rutting measures for new Iowa HMA pavements. However, the locally-calibrated rutting model provides better predictions than the nationally-calibrated model, which underestimates HMA layer rutting measures and overestimates granular base and subgrade layer rutting measures.
- The nationally calibrated alligator (bottom-up) cracking model provides acceptable predictions for new Iowa HMA pavement.
- The locally calibrated longitudinal (top-down) cracking model provides better predictions with lower bias and standard errors than the nationally calibrated model with severely underestimated longitudinal measures. Note that improved HMA longitudinal cracking models are being developed under NCHRP projects.

- Little or no thermal cracking was predicted using the MEPDG when using the proper binder grade for Iowa climate conditions, but significant thermal cracking measurements are observed in the field in new Iowa HMA pavement systems. Note that improved thermal cracking models have been developed through recently-completed FHWA pooled fund studies.
- Good agreement is observed between new Iowa HMA IRI measurements and predictions from (1) IRI model containing nationally calibrated distress model inputs with nationally calibrated model coefficients and (2) IRI model containing locally calibrated distress model inputs with nationally calibrated model coefficients.

Conclusions: HMA over JPCP

- Both nationally as well as locally calibrated rutting models provide good predictions of the total (accumulated) rutting measures for Iowa HMA over JPCP.
- Both nationally and locally calibrated alligator (bottom-up) cracking models provide acceptable predictions for Iowa HMA over JPCP.
- The locally calibrated longitudinal (top-down) cracking model provides better predictions with lower bias and standard errors than the nationally calibrated model with severely underestimated longitudinal cracking measures. Note that improved longitudinal cracking models are currently being developed through NCHRP projects.
- Transverse cracking measurements recorded in Iowa DOT PMIS do not differentiate between thermal cracking and reflection cracking measurements for HMA over JPCP. The current MEPDG reflection cracking model is purely empirical. The MEPDG thermal cracking model predicts little or no thermal cracking when using proper binder grade according to local climate condition. Note that mechanistic-based reflection cracking models have been developed through the recently completed NCHRP 1-41 project and improved HMA thermal cracking models have been developed through FHWA pooled-fund studies.
- Good agreements are observed between Iowa HMA over JPCP IRI measures and predictions from (1) IRI model of national calibrated distress inputs with national calibrated coefficients and (2) IRI model of local calibrated distress inputs with national calibrated coefficients.

Recommendations

- The locally calibrated JPCP performance prediction models (faulting, transverse cracking and IRI) identified in this study are recommended for use in Iowa as alternatives to their nationally calibrated counterparts.
- The locally calibrated rutting models identified in this study are recommended for use in HMA and HMA over JPCP systems as alternatives to the nationally calibrated ones.
- The nationally calibrated alligator (bottom-up) cracking models are recommended for use in Iowa HMA and HMA over JPCP systems.
- It is recommended to use MEPDG for longitudinal cracking, thermal cracking, and reflection cracking analysis in HMA and HMA overlay JPCP only for experimental/informational purposes, and not for decision making in design until these distress models (which are currently undergoing refinement) are fully implemented. Among these distress models, the M-E based reflection cracking model was developed through NCHRP 1-41 in which the ISU

research team (under Dr. Ceylan's supervision) played a key role. The local calibration of the NCHRP 1-41 reflection cracking model is recommended for rehabilitation design of HMA over JPCP in Iowa.

- It is recommended to use the nationally calibrated IRI model coefficients for Iowa HMA and HMA over JPCP systems because the HMA longitudinal cracking and thermal cracking models as IRI design inputs are still evolving and the accuracy of nationally calibrated IRI model is acceptable for Iowa conditions.
- The local calibration in this study is for the network level of pavement systems. It is recommended that Iowa DOT develop a satellite pavement management/pavement design database for each project being designed and constructed using the MEPDG/ DARWin-ME™ as part of the current PMIS. This database should be in comparable format to MEPDG/ DARWin-ME™ inputs and outputs. The database could be utilized to identify the cause of specific pavement failure in each project and do recalibration of MEPDG performance prediction models for non-traditional paving materials such as recycled materials, warm mix asphalt (WMA), etc.
- A simplified, cost-effective end-user design tool based on MEPDG/ DARWin-ME™ needs to be developed in order to enable not only Iowa DOT, but also county and city engineers to harness the benefits of DARWin-ME™ software development. A significant outcome of this tool will be the real-time generation of useful information that will help identify the effect of pavement design and construction features on future pavement performance without actually running the costly DARWin-ME™ by the end user. This could also be incorporated into a simplified Iowa pavement thickness design catalog as a guide for county, city and Iowa DOT engineers.
- Preliminary studies were carried out to see if there are any differences between the latest MEPDG (version 1.1) and DARWin-ME™ performance predictions for new JPCP, new HMA, and HMA over JPCP. The results indicated that the differences between the predictions of the two software versions are quite significant, at least in some cases warranting further investigation to determine if the local calibration study needs to be repeated using DARWin-ME™ solutions, which is now renamed as Pavement ME Design (version 1.3).

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APPENDIX A. LITERATURE REVIEW RESULTS

The national calibration-validation process was successfully completed for Mechanistic-Empirical Pavement Design Guide (MEPDG) (NCHRP 2004). Although this effort was comprehensive, a further validation study is highly recommended by the MEPDG as a prudent step in implementing a new design procedure that is so different from the current procedures. The objective of this task is to review all of available existing literature with regard to implementing the MEPDG and local calibration at national and local research levels. A comprehensive literature review was undertaken specifically to identify the following information:

- Identify local calibration steps detailed in national level research studies (National Cooperative Highway Research Program (NCHRP) and Federal Highway Administration (FHWA) research projects) for local calibration.
- Examine how State agencies apply the national level research projects' local calibration procedures in their pavement systems.
- Summarize MEPDG pavement performance models' local calibration coefficients reported in literature.

Summary of National Level Projects for MEPDG Local Calibration

AASHTO Guide for the Local Calibration of the ME PDG Developed from NCHRP Projects

At the request of the American Association of State Highway and Transportation Officials (AASHTO) Joint Task Force on Pavements (JTFP), the NCHRP initiated the project, 1-40 "*Facilitating the Implementation of the Guide for the Design of New and Rehabilitated Pavement Structures*" following NCHRP 1- 37A (NCHRP 2004) for implementation and adoption of the recommended MEPDG (TRB 2009). A key component of the NCHRP 1-40 is an independent, third-party review to test the design guide's underlying assumptions, evaluate its engineering reasonableness and design reliability, and to identify opportunities for its implementation in day-to-day design production work. Beyond this immediate requirement, NCHRP 1-40 includes a coordinated effort to acquaint state DOT pavement designers with the principles and concepts employed in the recommended guide, assist them with the interpretation and use of the guide and its software and technical documentation, develop step-by-step procedures to help State DOT engineers calibrate distress models on the basis of local and regional conditions for use in the recommended guide, and perform other activities to facilitate its acceptance and adoption.

There are two NCHRP research projects that are closely related to local calibration of MEPDG performance predictions. They are (1) NCHRP 9-30 project (NCHRP 2003a, NCHRP 2003b), "*Experimental Plan for Calibration and Validation of Hot Mix Asphalt Performance Models for Mix and Structural Design"* and (2) NCHRP 1-40B (Von Quintus et al. 2005, NCHRP 2007, Von Quintus et al. 2009a, Von Quintus et al. 2009b, NCHRP 2009, TRB 2010), "*User Manual and Local Calibration Guide for the Mechanistic-Empirical Pavement Design Guide and*

Software." Under the NCHRP 9-30 project, pre-implementation studies involving verification and recalibration have been conducted in order to quantify the bias and residual error of the flexible pavement distress models included in the MEPDG (Muthadi 2007). Based on the findings from the NCHRP 9-30 study, the NCHRP 1-40B project has focused on preparing (1) a user manual for the MEPDG and software and (2) detailed, practical guide for highway agencies for local or regional calibration of the distress models in the MEPDG and software. The manual and guide have been presented in the form of a draft AASHTO recommended practices; the guide shall contain two or more examples or case studies illustrating the step-by-step procedures. It was also noted that the longitudinal cracking model be dropped from the local calibration guide development in NCHRP 1-40B study due to lack of accuracy in the predictions (Muthadi 2007, Von Quintus and Moulthrop 2007). NCHRP 1-40 B was completed in 2009 and now published as "Guide for the Local Calibration of the Mechanistic-Empirical Pavement Design Guide" in AASHTO.

NCHRP 1-40B study (NCHRP 2007) initially provided the primary threes steps for calibrating MEPDG to local conditions and materials as follows:

Step. 1. *Verification of MEPDG performance models with national calibration factors*: Run the current version of the MEPDG software for new field sections using the best available materials and performance data. The accuracy of the prediction models was evaluated using the bias (defined as average over or under prediction) and the residual error (defined as the predicted minus observed distress) as illustrated in Figure A.1. If there is a significant bias and residual error, it is recommended to calibrate the models to local conditions leading to the second step.

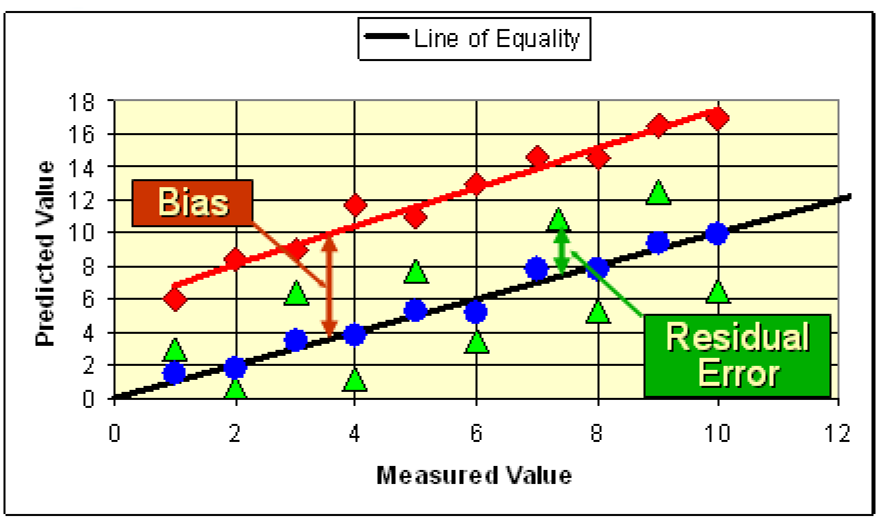


Figure A.1. The Bias and the residual error (Von Quintus 2008a)

Step. 2. *Calibration of the model coefficients*: eliminate the bias and minimize the standard error between the predicted and measured distresses.

Step. 3. *Validation of MEPDG performance models with local calibration factors*: Once the bias is eliminated and the standard error is within the agency's acceptable level after the calibration, validation is performed on the models to check for the reasonableness of the performance predictions.

NCHRP 1-40B study (NCHRP 2009) has also detailed these steps more into 11 steps for local calibration of the MEPDG. These 11 steps are depicted in Figure A.2Figure A.2. Flow chart for the procedure and steps suggested for local calibration: steps 1-5 (NCHRP 2009)

and Figure A.3**Error! Reference source not found.** below and each of the 11 steps is summarized in the following subsections.

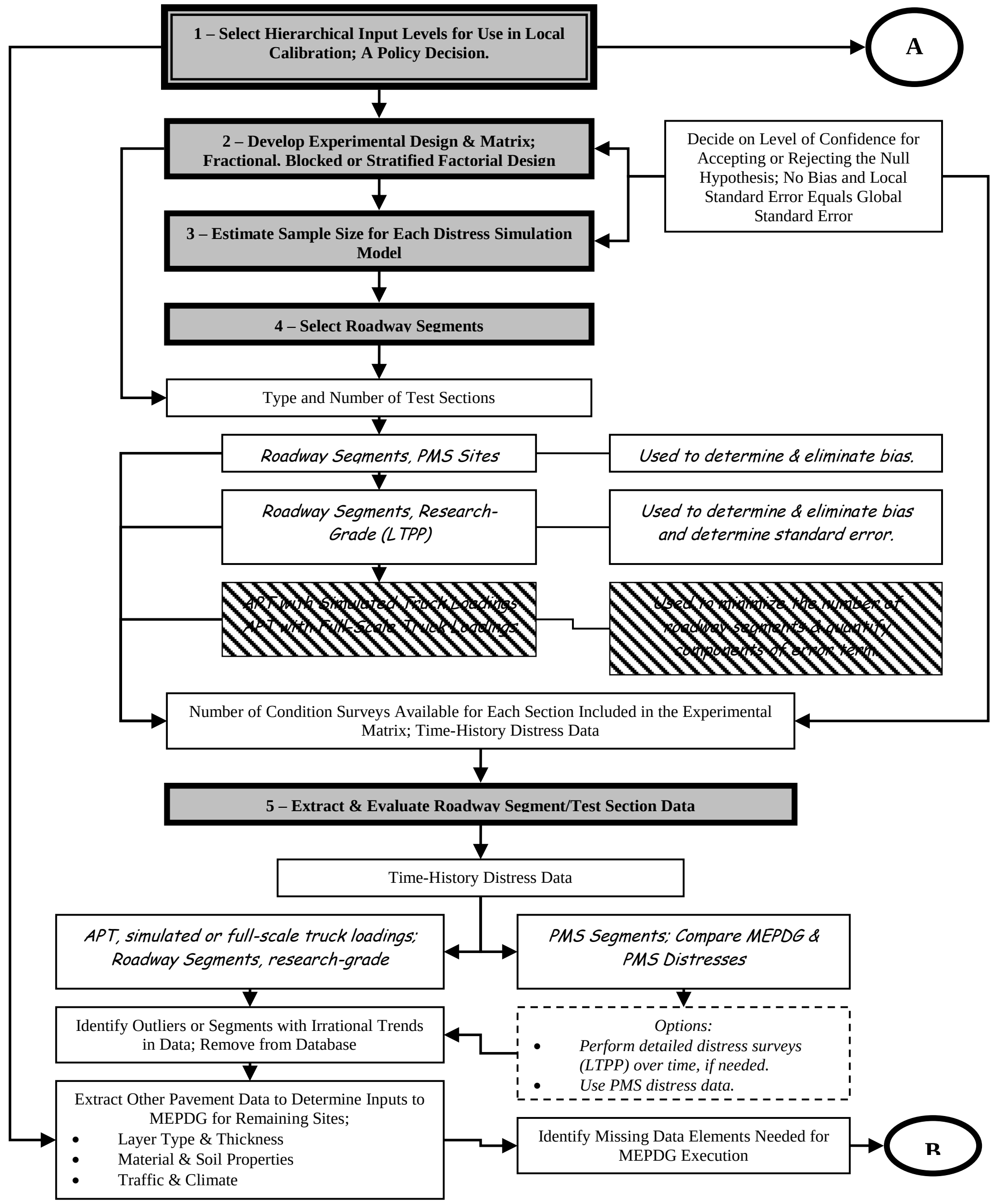


Figure A.2. Flow chart for the procedure and steps suggested for local calibration: steps 1-5 (NCHRP 2009)

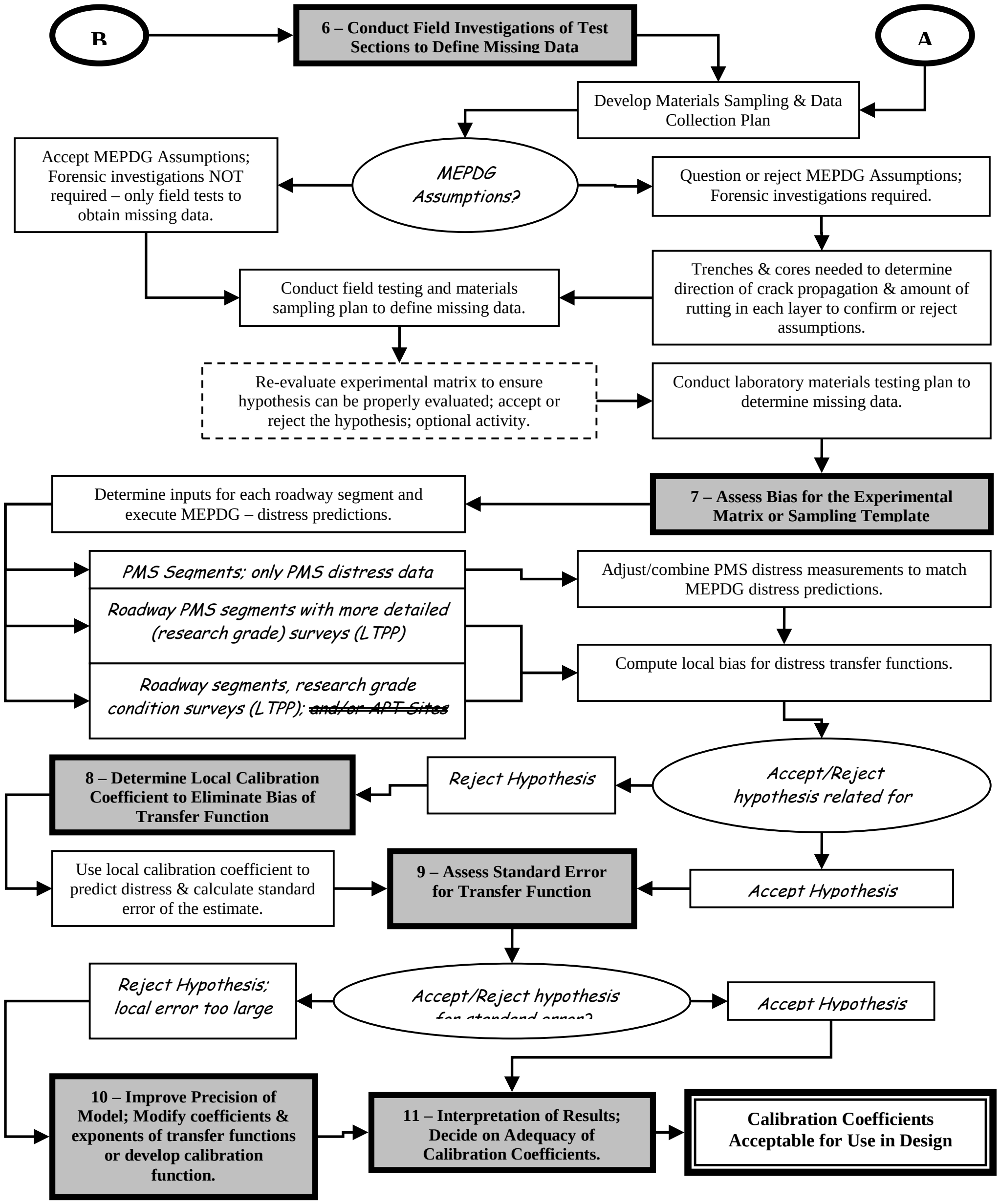


Figure A.3. Flow chart for the procedure and steps suggested for local calibration: steps 6-11 (NCHRP 2009)

<u>Step 1: Select Hierarchical Input Level</u>

The hierarchical input level to be used in the local validation-calibration process should be consistent with the way the agency intends to determine the inputs for day-to-day use. Some of input level 3 data could be available in the state Department of Transportation (DOT) pavement management system (PMS). It is also important to point out that the calibration using level 1 and 2 input data is dependent upon material and mixture characteristics. Further the linkage of material and mixture characteristics to pavement performance is critical to the level 1 and 2 calibrations. The general information from which the inputs were determined for each input category is discussed in Step 5.

Step 2: Experimental Factorial & Matrix or Sampling Template

A detailed sampling template should be created considering traffic, climate, pavement structure and materials representing local conditions. The number of roadway segments selected for the sampling template should result in a balanced factorial with the same number of replicates within each category.

Step 3: Estimate Sample Size for Each Performance Indicator Prediction Model

The sample size (total number of roadway segments or projects) can be estimated with statistical confidence level of significance. The selection of higher confidence levels can provide more reliable data but increase the number of segments needed. The number of distress observations per segment is dependent on the measurement error or within segment data variability over time (i.e.; higher the within project data dispersion or variability, larger the number of observations needed for each distress). The number of distress measurements made within a roadway segment is also dependent on the within project variability of the design features and site conditions. NCHRP 1-40B project report (NCHRP 2009) provides the following equation in determination of the number of distress observations:

$$N = \left(\frac{z_{\alpha}(s_y)}{e_t} \right)^2 \tag{A.1}$$

Where, z_{α} = 1.282 for a 90 percent confidence interval; s_y = standard deviation of the maximum true or observed values; and e_t = tolerable bias. The tolerable bias will be estimated from the levels that are expected to trigger some major rehabilitation activity, which are agency dependent. The s_e/s_y value (ratio of the standard error and standard deviation of the measured values) will also be agency dependent.

Step 4: Select Roadway Segments

Roadway segments should be selected to cover a range of distress values that are of similar ages within the sampling template. Roadway segments exhibiting premature or accelerated distress levels, as well as those exhibiting superior performance (low levels of distress over long periods of time), can be used, but with caution. The roadway segments selected for the sampling template when using hierarchal input level 3 should represent average performance conditions. It is important that the same number of performance observations per age per each roadway segment be available in selecting roadway segments for the sampling template. It would not be good practice to have some segments with ten observations over 10 years with other segments having only two or three observations over 10 years. The segments with one observation per year would have a greater influence on the validation-calibration process than the segments with less than one observation per year.

<u>Step 5: Extract and Evaluate Roadway Segment/Test Section Data</u>

This step is grouped into four activities: (1) extracting and reviewing the performance data; (2) comparing the performance indicator magnitudes to the trigger values; (3) evaluating the distress data to identify anomalies and outliers; and (4) determining the inputs to the MEPDG. First, measured time-history distress data should be made from accelerated pavement testing (APT) or extracted from agency PMS. The extraction of data from agency PMS should require a prior step of reviewing PMS database to determine whether the measured values are consistent with the values predicted by the MEPDG. NCHRP 1-40B project report (NCHRP 2009) demonstrated the conversion procedures of pavement distress measurement units between PMS and MEPDG for flexible pavements PMS database of Kansas Department of Transportation (KSDOT) and rigid pavements PMS database of Missouri Department of Transportation (MODOT). These examples in NCHRP 1-40B project report (NCHRP 2009) is reproduced in below.

For the flexible pavement performance data in KSDOT, the measured cracking values are different, while the rutting and International Roughness Index (IRI) values are similar and assumed to be the same. The cracking values and how they were used in the local calibration process are defined below.

- *Fatigue Cracking.* KSDOT measures fatigue cracking in number of wheel path feet per 100 foot sample by crack severity, but do not distinguish between alligator cracking and longitudinal cracking in the wheel path. In addition, reflection cracks are not distinguished separately from the other cracking distresses. The PMS data were converted to a percentage value similar to what is reported in the Highway Performance Monitoring System (HPMS) system from Kansas. In summary, the following equation was used to convert KSDOT cracking measurements to a percentage value that is predicted by the MEPDG.

$$FC = \left(\frac{FCR_1(0.5) + FCR_2(1.0) + FCR_3(1.5) + FCR_4(2.0)}{8.0} \right) \tag{A.2}$$

  All load related cracks are included in one value. Thus, the MEPDG predictions for load related cracking were combined into one value by simply adding the length of longitudinal cracks and reflection cracks for Hot Mix Asphalt (HMA) overlays, multiplying by 1.0 ft, dividing that product by the area of the lane and adding that value to the percentage of alligator cracking predicted by the MEPDG.

- *Transverse Cracking.* Another difference is that KSDOT records thermal or transverse cracks as the number of cracks by severity level. The following equation has been used by KSDOT to convert their measured values to the MEPDG predicted value of ft./mi.

$$TC = \left(\frac{TCR_o + TCR_1 + TCR_2 + TCR_3}{(10)(12)(52.8)} \right) \quad (A.3)$$

The value of 10 in the above equation is needed because the data are stored with an implied decimal. The value of 12 ft is the typical lane width, and the value of 52.8 coverts from 100 foot sample to a per mile basis. Prior to 1999, KSDOT did not record the number or amount of sealed transverse cracking (TCR_0). As a result, the amount of transverse cracks sometimes goes to “0”.

For the rigid pavement performance data in MODOT, the measured transverse cracking values are different from MEPDG, while the transverse joint faulting and IRI values are similar and assumed to be the same. The transverse cracking values and how they were used in the local calibration process are defined below.

- *Transverse Cracking.* MEPDG requires the percentage of all Portland Cement Concrete (PCC) slabs with mid panel fatigue transverse cracking. Both MOODT and LTPP describe transverse cracking as cracks that are predominantly perpendicular to the pavement slab centerline. Measured cracking is reported in 3 severity levels (low, medium, and high) and provides distress maps showing the exact location of all transverse cracking identified during visual distress surveys. Thus, the databases contain, for a given number of slabs within a 500-ft pavement segment, the total number of low, medium, and high severity transverse cracking. Since LTPP does not provide details on whether a given slab has multiple cracks, as shown in Figure A.4**Error! Reference source not found.**, a simple computation of percent slabs with this kind of data can be misleading. Therefore, in order to produce an accurate estimate of percent slab cracked, distress maps or videos prepared as part of distress data collection were reviewed to determine the actual number of slabs with transverse “fatigue” cracking for the 500-ft pavement segments. Total number of slabs was also counted. Percent slabs cracked was defined as follows:

$$Percent\ Slabs\ Cracked = \left(\frac{Number\ of\ cracked\ slabs}{Total\ number\ of\ slabs} \right) * 100 \quad (A.4)$$

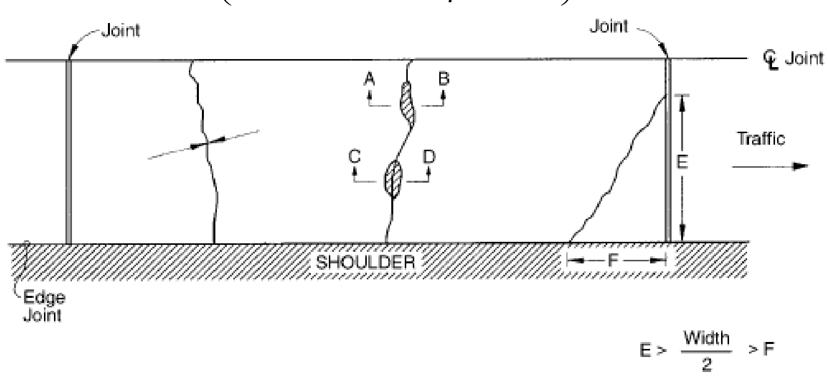


Figure A.4. LTPP transverse cracking (Miller and Bellinger 2003)

- *Transverse Joint Faulting.* It is measured and reported by MODOT and LTPP as the difference in elevation to the nearest 1 mm between the pavement surfaces on either side of a transverse joint. The mean joint faulting for all joints within a 500-ft pavement section is reported. This is comparable to the MEPDG predicted faulting.

- *IRI.* The values included in the MODOT PMS database are comparable to the MEPDG predicted IRI.

The second activity of step 5 is to compare the distress magnitudes to the trigger values for each distress. In other words, answer the question—Does the sampling template include values close to the design criteria or trigger value? This comparison is important to provide answer if the collected pavement distress data could be properly utilized to validate and accurately determine the local calibration values. For example, low values of fatigue cracking measurements comparing to agency criteria is difficult to validate and accurately determine the local calibration values or adjustments for predicting the increase in cracking over time.

The distress data for each roadway segment included in the sampling template should be evaluated to ensure that the distress data are reasonable time-history plots. Any zeros that represent non-entry values should be removed from the local validation-calibration database. Distress data that return to zero values within the measurement period may indicate some type of maintenance or rehabilitation activity. Measurements taken after structural rehabilitation should be removed from the database or the observation period should end prior to the rehabilitation activity. Distress values that are zero as a result of some maintenance or pavement preservation activity, which is a part of the agency's management policy, should be removed but future distress observation values after that activity should be used. If the outliers or anomalies of data can be explained and are a result of some non-typical condition, they should be removed. If the outlier or anomaly cannot be explained, they should remain in the database.

The MEPDG pavement input database related to each selected roadway segment should be prepared to execute MEPDG software. The existing resource of these input data for level 3 analyses are agency PMS, traffic database, as-built plans, construction database files and etc. If adequate data for level 3 were unavailable, the mean value from the specifications was used or the average value determined for the specific input from other projects with similar condition. The default values of MEPDG could also be utilized in this case.

<u>Step 6: Conduct Field and Forensic Investigations</u>

Field and forensic investigations could be conducted to check the assumptions and conditions included in the MEPDG for the global (national) calibration effort. These field and forensic investigations include measuring the rutting in the individual layers, determining where the cracks initiated or the direction of crack propagation, and determining permanent curl/warp effective temperature and etc. The field and forensic investigations is not necessary if agency accepts the assumptions and conditions included in the MEPDG.

Step 7: Assess Local Bias from Global Calibration Factors

The MEPDG software is executed using the global calibration values to predict the performance indicators for each roadway segment selected. The null hypothesis is first checked for the entire sampling matrix. The null hypothesis in equation below is that the average residual error ($e_r = y_{Measured} - x_{predicted}$) or bias is zero for a specified confidence level or level of significance.

$$H_O : \sum_{i=1}^{n} \left(y_{Measured} - x_{\Pr edicted} \right)_i = 0 \qquad \text{(A.5)}$$

It is helpful for assessment through making plots of a comparison between the predicted ($x_{predicted}$) and the measured values ($y_{Measured}$) and a comparison between the residual errors (e_r) and the predicted values ($x_{predicted}$) for each performance indicator (See Figure A.5).

Two other model parameters can be also used to evaluate model bias—the intercept (b_o) and slope (m) estimators using the following fitted linear regression model between the measured ($y_{Measured}$) and predicted ($x_{predicted}$) values.

$$\hat{y}_i = b_o + m(x_i) \qquad \text{(A.6)}$$

The intercept (b_o) and slope (m) estimators can provide not only accuracy quantity of each prediction but also identification of dependent factors such as pavement structure (new construction versus rehabilitation) and HMA mixture type (conventional HMA versus Superpave mixtures) to each prediction. For illustration, **Error! Reference source not found.** presents comparison of the intercept and slope estimators to the line of equality for the predicted and measured rut depths using the global calibration values.

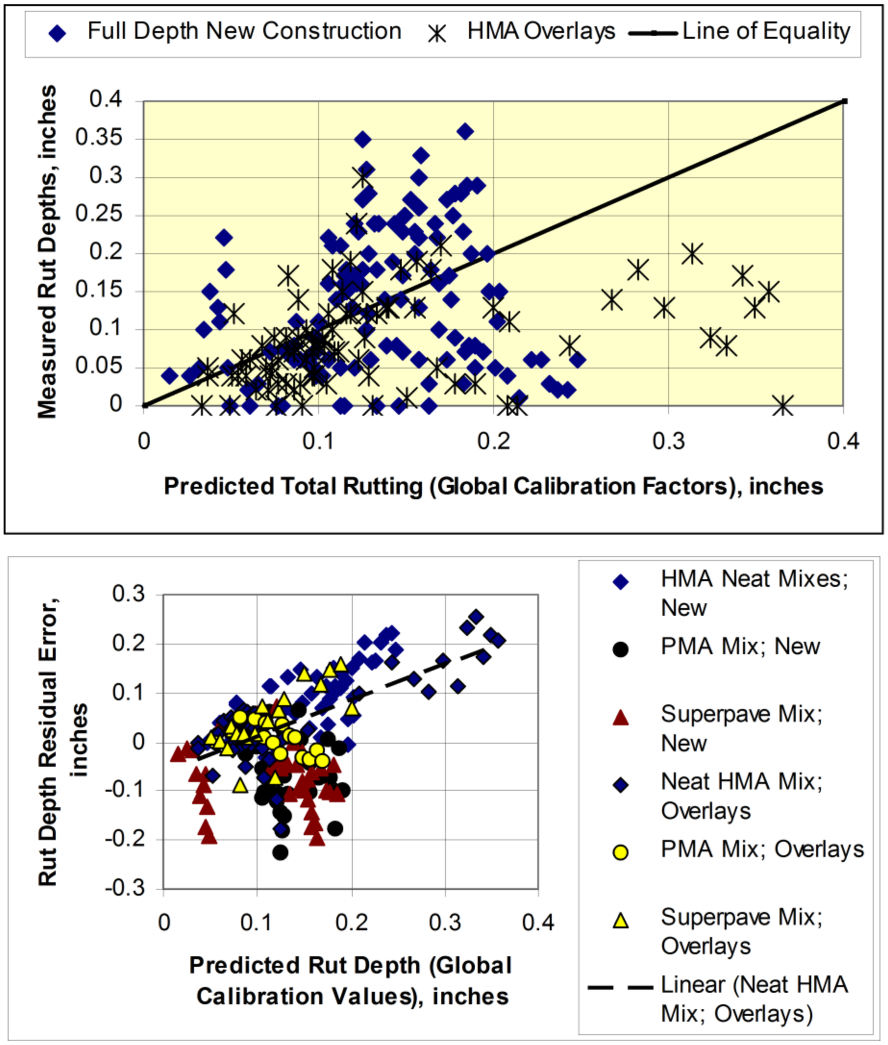
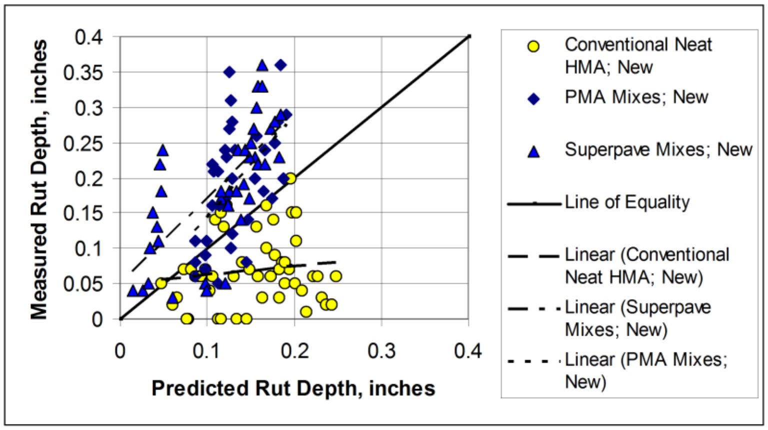
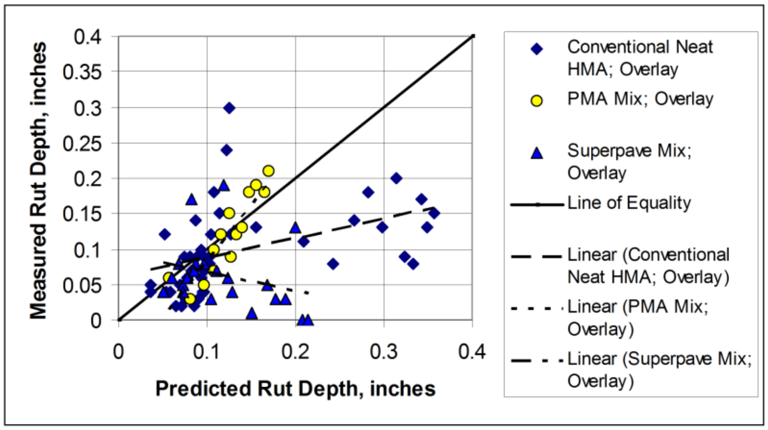


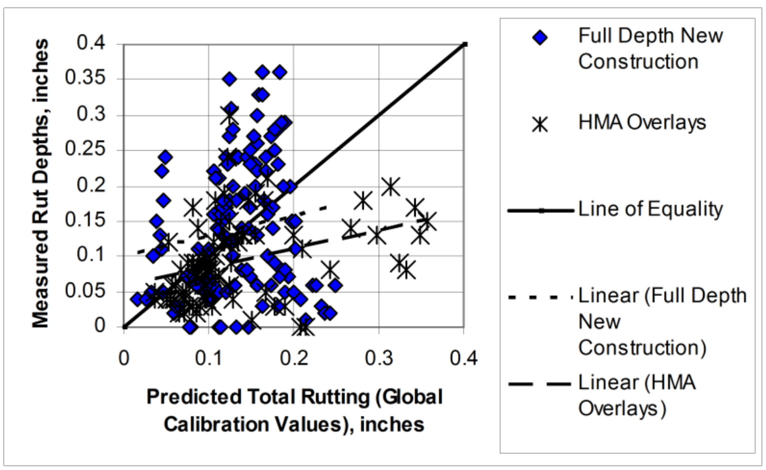
Figure A.5. Comparison of predicted and measured rut depths using the global calibration in KSDOT study (NCHRP 2009)



a. Intercept and slope estimators that are dependent on mixture type for the new construction PMS segments.



b. Intercept and slope estimators that are dependent on mixture type for the rehabilitation PMS segments.



c. Intercept and slope estimators that are structure dependent for the PMS segments.

Figure A.6. Comparison of the intercept and slope estimators to the line of equality for the predicted and measured rut depths using the global calibration values in KSDOT study (NCHRP 2009)

Step 8: Eliminate Local Bias of Distress Prediction Models

The MPEDG software includes two sets of parameters for local calibration of most performance indicator transfer functions. One set is defined as agency specific values and the other set as local calibration values. Figure A.7 shows a screen shot of the tools section where these values can be entered into the software for each performance indicator on a project basis. The default values of MEPDG performance indicator transfer functions are global calibration values for agency specific values (k_1, k_2, and k_3 in Figure A.7) and are one for local calibration values (β_1, β_2, and β_3 in Figure A.7). These parameters are used to make adjustments to the predicted values so that the difference between the measured and predicted values, defined as the residual error, is minimized. Either one can be used with success. Appendix A presents screen shots of the MEPDG software (Version 1.1) tools section for all of performance indicators of rehabilitated HMA pavement and new PCC pavement.

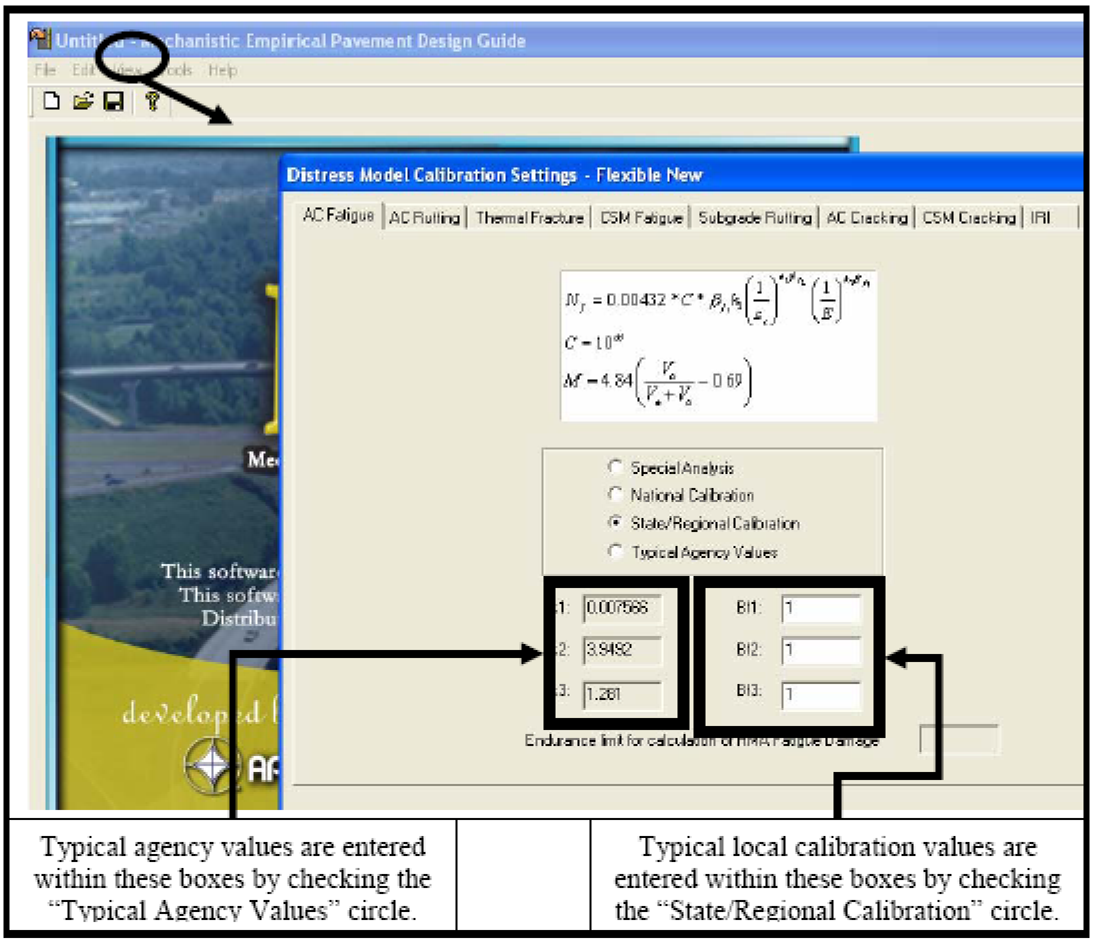


Figure A.7. Screen Shot of the MEPDG Software for the local calibration and agency specific values (Von Quintus 2008b)

NCHRP 1-40B project study (2009) lists the coefficients of the MEPDG transfer functions or distress and IRI prediction models that should be considered for revising the predictions to eliminate model bias for flexible pavements and HMA overlays. **Error! Reference source not found.** from NCHRP 1-40B project study (2009) was prepared to provide guidance in eliminating any local model bias in the predictions. The distress specific parameters can be dependent on site factors, layer parameters, or policies of the agency.

Table A.1. Calibration parameters to be adjusted for eliminating bias and reducing the standard error of the flexible pavement transfer functions (NCHRP 2009)

(a) HMA pavements

Distress	Eliminate Bias	Reduce Standard Error
Rutting	k_{r1}, β_{s1} or β_{r1}	k_{r2}, k_{r3}, and β_{r2}, β_{r3}
Alligator cracking	C_2 or k_{f1}	k_{f2}, k_{f3}, and C_1
Longitudinal cracking	C_2 or k_{f1}	k_{f2}, k_{f3}, and C_1
Load related cracking – semi-rigid pavements	C_2 or β_{c1}	C_1, C_2, and C_4
Thermal cracking	β_{t3}	β_{t3}
IRI	C_4	C_1, C_2, and C_3

(b) PCC pavements

Distress	Eliminate Bias	Reduce Standard Error
Faulting	C_1	$C_2 – C_8$
JPCP transverse cracking	C_1 or C_4	C_2 and C_5
CRCP fatigue cracking	C_1	C_2
CRCP punchouts	C_3	C_4 and C_5
CRCP crack widths	C_6	C_6
JPCP IRI	C_4	C_1
CRCP IRI	C_4	C_1 and C_2

The process to eliminate the bias is applied to the globally calibrated pavement performance transfer functions found to result in bias from step 7. The process used to eliminate the bias depends on the cause of that bias and the accuracy desired by the agency. NCHRP 1-40B project study (NCHRP 2009) addresses three possibilities of bias and the bias elimination procedures corresponding to each possibility reproduced below.

1. The residual errors are, for the most part, always positive or negative with a low standard error of the estimate in comparison to the trigger value, and the slope of the residual errors versus predicted values is relatively constant and close to zero. In other words, the precision of the prediction model is reasonable but the accuracy is poor. In this case, the local calibration coefficient is used to reduce the bias. This condition generally requires the least level of effort and the fewest number of runs or iterations of the MEPDG with varying the local calibration values to reduce the bias. The statistical assessment described in step 7 should be conducted to the local calibrated pavement performance to check obtaining agency acceptable bias.
2. The bias is low and relatively constant with time or number of loading cycles, but the residual errors have a wide dispersion varying from positive to negative values. In other words, the accuracy of the prediction model is reasonable, but the precision is poor. In this case, the coefficient of the prediction equation is used to reduce the bias but the value of the local calibration coefficient is probably dependent on some site feature, material property, and/or design feature included in the sampling template. This condition generally requires more runs and a higher level of effort to reduce dispersion of the residual errors. The statistical assessment described in step 7 should be conducted to the local calibrated pavement performance to check obtaining agency acceptable bias.
3. The residual errors versus the predicted values exhibit a significant and variable slope that is dependent on the predicted value. In other words, the precision of the prediction model is poor and the accuracy is time or number of loading cycles dependent—there is poor correlation between the predicted and measured values. This condition is the most difficult to evaluate because the exponent of the number of loading cycles needs to be considered. This condition also requires the highest level of effort and many more MEPDG runs with varying the local calibration values to reduce bias and dispersion. The statistical assessment described in step 7 should be conducted to the local calibrated pavement performance to check obtaining agency acceptable bias.

Step 9: Assess Standard Error of the Estimate

After the bias was reduced or eliminated for each of the transfer functions, the standard error of the estimate (SEE, s_e) from the local calibration is evaluated in comparison to the SEE from the global calibration. The standard error of the estimate for each globally calibrated transfer function is included under the “Tools” section of the MEPDG software. Figure A.8 illustrates the comparison of the SEE for the globally calibrated transfer functions to the SEE for the locally calibrated transfer functions.

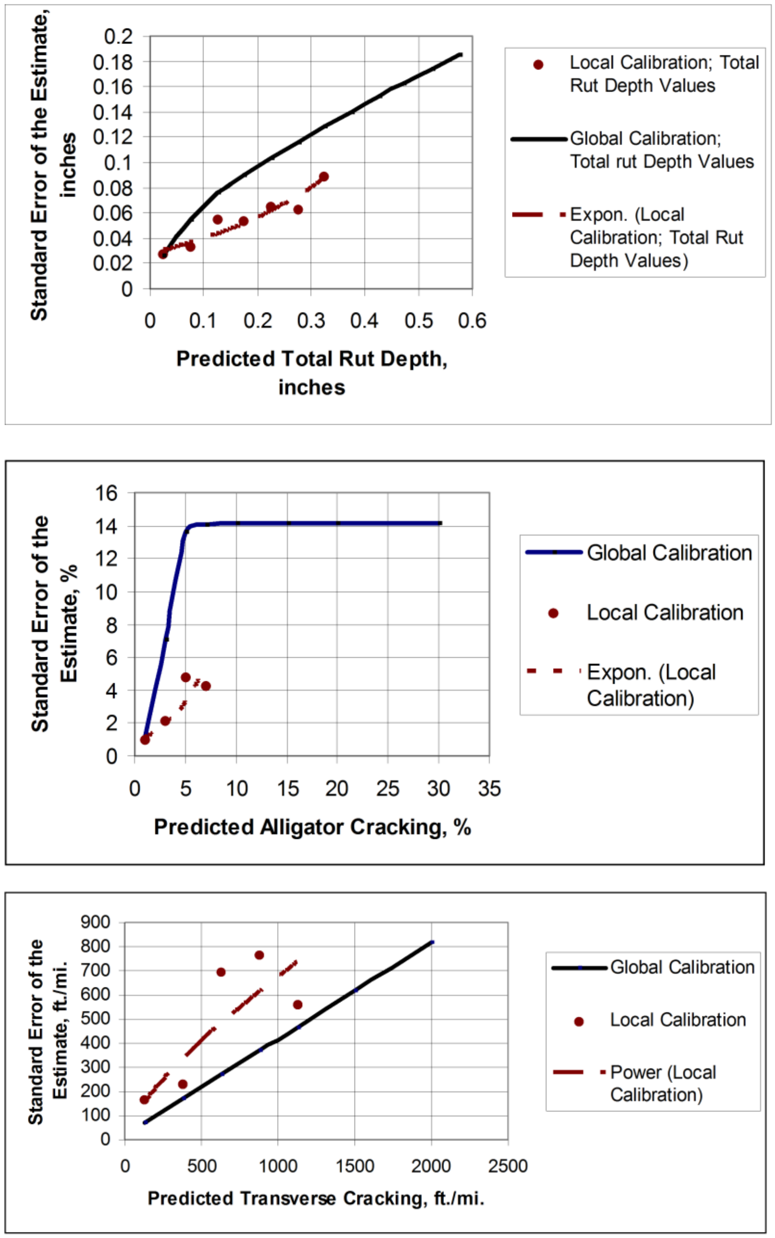


Figure A.8. Comparison of the standard error of the estimate for the global-calibrated and local-calibrated transfer function in KSDOT study (NCHRP 2009)

<u>Step 10: Reduce Standard Error of the Estimate</u>

If the SEE from the local calibration is found in step 9 to be statistically different in comparison to the SEE included in the MEPDG for each performance indicator, an statistical analysis of variance (ANOVA) can be conducted to determine if the residual error or bias is dependent on some other parameter or material/layer property for the selected roadway segments. If no correlation would be identified, the local calibration factors determined from step 8 and the SEE values obtained from step 9 could be considered as the final products for the selected roadway segments. If some correlation to some parameters (for example, HMA mixture volumetric properties) would be identified, the local calibration values should be determined for each type in correlated parameters or new calibration function should be developed. NCHRP Project 1-40B and Von Quintus (2008b) documented HMA mixture specific factors used to modify or adjust the MEPDG global calibration factors for the rut depth and the bottom-up cracking transfer functions where sufficient data are available.

Step 11: Interpretation of Results and Deciding on Adequacy of Calibration Factors

The purpose of this step is to decide whether to adopt the local calibration values or continue to use the global values that were based on data included in the LTPP program from around the U.S. To make that decision, an agency should identify major differences between the LTPP projects and the standard practice of the agency to specify, construct, and maintain their roadway network. More importantly, the agency should determine whether the local calibration values can explain those differences. The agency should evaluate any change from unity for the local calibration parameters to ensure that the change provides engineering reasonableness.

FHWA Projects

Two research study supported by FHWA have been conducted to use pavement management information system (PMIS) data for local calibration of MEPDG. One is "*Using Pavement Management Data to Calibrate and Validate the New MEPDG, An Eight State Study* (FHWA 2006a, FHWA 2006b)." This study evaluated the potential use of PMIS on MEPDG calibrations from eight participated states: Florida, Kansas, Minnesota, Mississippi, New Mexico, North Carolina, Pennsylvania, and Washington. The study concluded that all the participating states could feasibly use PMIS data on MEPDG calibrations and others states not participating in this study could also do. It is recommended that each SHA should develop a satellite pavement management/pavement design database for each project being designed and constructed using the MEPDG in part of current PMIS used.

As following previous one, *FHWA HIF-11-026 research project the local calibration of MEPDG using pavement management system* (FHWA 2010a, FHWA 2010b) was conducted to develop a framework for using existing PMIS to calibrate the MEPDG performance model. One state (North Carolina) was selected from screening criteria to finalize and verify the MEPDG calibration framework based on the set of actual conditions. As following developed framework, local calibration of a selected state was demonstrated under the assumptions of both MEPD performance predictions established from NCHRP 1-37 A and distress measurements from a selected state. Note that NC DOT used **subjective distress rating** with severity in accordance to state DOT manual rather than LTPP manual. Table A.2 listed the assumptions used for MEPDG local calibration in this study.

Table A.2. List of assumptions in MEPDG local calibration of NC under FHWA HIF-11-026 research project (FHWA 2010)

Type	**Performance Predictions[1]**	**Assumptions**
HMA	Rutting	• Rutting measurement was assumed to progress from zero to the assumed numeric value over the life of the pavement in order to convert NCDOT subjective rut rating into an estimated measured value. ✓ Low severity – 0.5 in. (12.7 mm). ✓ Moderate severity – 1.0 in. ✓ High severity – Not applicable • Rut depth progression was based on the number of NCDOT rut depth ratings and distributed over the measurement period to best reflect the slope of the MEPDG predicted rut depth over time. • For HMA overlay, the rut condition prior to the applied overlay was selected.
	Alligator Cracking	• A sigmoid function form of MEPDG alligator cracking is the best representation of the relationship between cracking and damage. The relationship must be “bounded” by 0 ft^2 cracking as a minimum and 6,000 ft^2 cracking as a maximum[2]. • Alligator cracking is to 50 percent cracking of the total area of the lane (6000 ft^2) at a damage percentage of 100 percent[2]. • Since alligator cracking is related to loading and asphalt layer thickness, alligator crack prediction is similar for a wide range of temperatures[2]. • All load-related cracking was considered to initiate from the bottom up (alligator cracking). • The alligator cracking measurement was estimated from tensile strains at the bottom of the asphalt layer calculated from a layer elastic analysis program by inputting MEPDG asphalt dynamic modulus corresponding to the NCDOT measured alligator distress rating. • The estimated alligator cracking measurement was distributed over the age of the pavement section.

Type	Performance Predictions[1]	Assumptions
	Thermal Cracking	• The model will not predict thermal cracking on more than 50 percent of the total section length[2]. • The maximum length of thermal cracking is 4224 ft/mi (400 ft/500 ft × 5280 ft/1mi) [2]. • Cracks were assumed to be full-lane width (i.e., 12 ft) for all severity levels. • For each pavement section, the section length was divided by the reported NCDOT cracking frequency and multiplied by the crack length (assumed to be 12 ft) to obtain the total estimated crack length per pavement section. • As with rutting and alligator cracking, the distress severity from the last NCDOT survey was used to calculate the thermal cracking numeric value.
JPCP	Transverse Cracking	• JPCP in NCDOT was assumed to be designed on average perform to the selected design criteria (15 percent slab cracking) at the specified reliability (90 percent). • The layer properties for these design runs were selected primarily as default values, as were most of the traffic characteristics.
	Faulting	• The layer properties for these design runs were selected primarily as default values, as were most of the traffic characteristics.

[1]Longitudinal cracking, reflection cracking, and smoothness were not considered in calibration due to lack of data and deficiency of model.
[2] The assumptions made from MEPD performance models in NCHRP 1-37 A.

State-Level/Local MEPDG Calibration Studies

As apart to national level projects, multiple state-level research efforts have been being conducted regarding the local calibration of the MEPDG involving each step described in NCHRP 1-40B study. However, not many research studies for MEPDG validation in local sections have been finalized because the MEPDG has constantly been updated through NCHRP projects (2006a; 2006b) after the release of the initial MEPDG software (Version 0.7). This section summarizes up to date MEPDG local calibration research efforts at the State level.

Flexible Pavements

A study by Galal and Chehab (2005) in Indiana compared the distress measures of existing HMA overlay over a rubblized PCC slab section using AASHTO 1993 design with the MEPDG (Version 0.7) performance prediction results using the same design inputs. The results indicated that MEPDG provide good estimation to the distress measure except top–down cracking. They also emphasized the importance of local calibration of performance prediction models.

Montana DOT conducted the local calibration study of MEPDG for flexible pavements (Von Quintus and Moulthrop 2007). In this study, results from the NCHRP 1-40B (Von Quintus et al. 2005) verification runs were used to determine any bias and the standard error, and compare that error to the standard error reported from the original calibration process that was completed under NCHRP Project 1-37A (NCHRP 2004). Bias was found for most of the distress transfer functions. National calibration coefficients included in Version 0.9 of the MEPDG were used initially to predict the distresses and smoothness of the Montana calibration refinement test sections to determine any prediction model bias. These runs were considered a part of the validation process, similar to the process used under NCHRP Projects 9-30 and 1-40B.The findings from this study are summarized for each performance model as shown below:

- Rutting prediction model: the MEPDG over-predicted total rut depth because significant rutting was predicted in unbound layers and embankment soils.
- Alligator cracking prediction model: the MEPDG fatigue cracking model was found to be reasonable.
- Longitudinal cracking prediction model: no consistent trend in the predictions could be identified to reduce the bias and standard error, and improve the accuracy of this prediction model. It is believed that there is a significant lack-of-fit modeling error for the occurrence of longitudinal cracks.
- Thermal cracking prediction model: the MEPDG prediction model with the local calibration factor was found to be acceptable for predicting transverse cracks in HMA pavements and overlays in Montana.
- Smoothness prediction model: the MEPDG prediction equations are recommended for use in Montana because there are too few test sections with higher levels of distress in Montana and adjacent States to accurately revise this regression equation.

Von Quintus (2008b) summarized the flexible pavement local calibration value results of the MEPDG from NCHRP project 9-30, 1-40 B, and Montana DOT studies listed in Table A.3. These results originally from Von Quintus (2008b) present in Table A.4 to Table A.6 for the rut depth, fatigue cracking, and thermal cracking transfer functions. These could be useful reference for states having similar conditions of studied sites. The detailed information of studied sites is described in Von Quintus (2008b).

Table A.3. Listing of local validation-calibration projects (Von Quintus 2008b)

Project Identification	Transfer Functions Included in the Local Validation and/or Calibration Efforts for Each Project				
	Rut Depths	Area Cracking	Longitudinal Cracking	Thermal Cracking	Smoothness or IRI
NCHRP Projects 9-30 & 1-40B; *Local Calibration Adjustments for HMA Distress Prediction Models in MEPDG Software*, (Von Quintus, et al., 2005a & b)	√	√	√		
Montana DOT, *MEPDG Flexible Pavement Performance Prediction Models for Montana*, (Von Quintus & Moulthrop, 2007a & b)	√	√	√	√	√
NCHRP Project 1-40B, *Examples Using Recommended Practice for Local Calibration of MEPDG Software*, Kansas Pavement Management Data, (Von Quintus, et al., 2008b)	√	√		√	√
NCHRP Project 1-40B, *Examples Using Recommended Practice for Local Calibration of MEPDG Software*, LTPP SPS-1 and SPS-5 Projects, (Von Quintus, et al., 2008b)	√	√		√	√

Table A.4. Summary of local calibration values for the rut depth transfer function (Von Quintus 2008b)

<table>
<tr><th colspan="2" rowspan="2">Project Identification</th><th colspan="2">Unbound Materials/Soils, β_{s1}</th><th colspan="3">HMA Calibration Values</th></tr>
<tr><th>Fine-Grained</th><th>Coarse-Grained</th><th>β_{r1}</th><th>β_{r3}</th><th>β_{r2}</th></tr>
<tr><td colspan="2" rowspan="2">NCHRP Projects 9-30 & 1-40B; Verification Studies, Version 0.900 of the MEPDG.</td><td>0.30</td><td>0.30</td><td colspan="3">Values dependent on volumetric properties of HMA; the values below represent the overall range.</td></tr>
<tr><td colspan="2">Insufficient information to determine effect of varying soil types.</td><td>6.9 to 10.8</td><td>0.65 to 0.90</td><td>0.90 to 1.10</td></tr>
<tr><td colspan="2" rowspan="2">Montana DOT; Based on version 0.900 of the MEPDG</td><td rowspan="2">0.30</td><td rowspan="2">0.30</td><td colspan="3">Values dependent on the volumetric properties of HMA; the values below represent overall averages.</td></tr>
<tr><td>7.0</td><td>0.70</td><td>1.13</td></tr>
<tr><td colspan="2">Kansas DOT; PM Segments; HMA Overlay Projects; All Mixtures (Version 1.0)</td><td>0.50</td><td>0.50</td><td>1.5</td><td>0.95</td><td>1.00</td></tr>
<tr><td rowspan="3">Kansas PM Segments; New Construction</td><td>Convent.</td><td rowspan="3">0.50</td><td rowspan="3">0.50</td><td>1.5</td><td>0.90</td><td>1.00</td></tr>
<tr><td>Superpave</td><td>1.5</td><td>1.20</td><td>1.00</td></tr>
<tr><td>PMA</td><td>2.5</td><td>1.15</td><td>1.00</td></tr>
<tr><td colspan="2" rowspan="2">LTPP SPS-1 & SPS-5 Projects built in accordance with specification; conventional HMA mixtures (Version 1.0).</td><td rowspan="2">0.50</td><td rowspan="2">0.50</td><td colspan="2">Value dependent on the air void & asphalt content</td><td>1.00</td></tr>
<tr><td>1.25 to 1.60</td><td>0.90 to 1.15</td><td>1.00</td></tr>
<tr><td colspan="2" rowspan="2">LTPP SPS-1 Projects with anomalies or construction difficulties, unbound layers.</td><td colspan="2">Values dependent on density and moisture content; values below represent the range found.</td><td rowspan="2">---</td><td rowspan="2">---</td><td rowspan="2">---</td></tr>
<tr><td>0.50 to 1.25</td><td>0.50 to 3.0</td></tr>
</table>

Table A.5. Summary of local calibration values for the area fatigue cracking transfer function (Von Quintus 2008b)

Project Identification		β_{f1}	β_{f2}	β_{f3}	C_2
NCHRP Projects 9-30 & 1-40B; Verification Studies, Version 0.900 of the MEPDG		Values dependent on the volumetric properties.			
		0.75 to 10.0	1.00	0.70 to 1.35	1.0 to 3.0
Montana DOT; Based on version 0.900 of the MEPDG, with pavement preservation treatments		Values dependent on the volumetric properties.			
		13.21	1.00	1.25	1.00
Northwest Sites; Located in States Adjacent to Montana, without pavement preservation treatments		Values dependent on the volumetric properties.			
		1.0 to 5.0	1.00	1.00	1.0 to 3.0
Kansas DOT; PM Segments; HMA Overlay Projects; All HMA Mixtures		0.05	1.00	1.00	1.00
Kansas DOT; PM Segments; New Construction	Conventional HMA Mixes	0.05	1.00	1.00	1.00
	PMA	0.005	1.00	1.00	1.00
	Superpave	0.0005	1.00	1.00	1.00
Mid-West Sites	LTPP SPS-1 Projects built in accordance with specifications	0.005	1.00	1.00	1.00
	LTPP SPS-1 Projects with anomalies or production difficulties	1.00	1.00	1.00	1.0 to 4.0
	LTPP SPS-5 Projects; Debonding between HMA Overlay and Existing Surface	0.005	1.00	1.00	1.0 to 4.0

Table A.6. Summary of the local calibration values for the thermal cracking transfer function (Von Quintus 2008b)

Project Identification		β_{t1}	β_{t2}	β_{t3}
Montana DOT; application of pavement preservation treatments.		---	---	0.25
Northwest Sites, located in states adjacent to Montana, but without pavement preservation treatments; appears to be agency dependent.		---	---	1.0 to 5.0
Kansas PM Segments; Full-Depth Projects	PMA	---	---	2.0
	Conventional	---	---	2.0
	Superpave	---	---	3.5
Kansas PMS Segments; HMA Overlay Projects	PMA	---	---	2.0
	Conventional	---	---	7.5
	Superpave	---	---	7.5
LTPP Projects; HMA produced in accordance with specifications	Conventional	---	---	Dependent on Asphalt Content & Air Voids
LTPP Projects; Severely aged asphalt	Conventional	---	---	7.5 to 20.0

Kang et al. (2007) prepared a regional pavement performance database for a Midwest implementation of the MEPDG. They collected input data required by the MEPDG as well as measured fatigue cracking data of flexible and rigid pavements from Michigan, Ohio, Iowa and Wisconsin State transportation agencies. They reported that the gathering of data was labor-intensive because the data resided in various and incongruent data sets. Furthermore, some pavement performance observations included temporary effects of maintenance and those observations must be removed through a tedious data cleaning process. Due to the lack of reliability in collected pavement data, the calibration factors were evaluated based on Wisconsin data and the distresses predicted by national calibration factors were compared to the field collected distresses for each state except Iowa. This study concluded that the default national calibration values do not predict the distresses observed in the Midwest. The collection of more reliable pavement data is recommended for a future study.

Schram and Abdelrahman (2006) attempted to calibrate two of MEPDG IRI models for the Jointed Plain Concrete Pavement (JPCP) and the HMA overlays of rigid pavements at the local project-level using Nebraska Department of Roads (NDOR) pavement management data. The focused dataset was categorized by annual daily truck traffic (ADTT) and surface layer thickness. Three categories of ADTT were considered: low (0 – 200 trucks/day), medium (201 – 500 trucks/day), and high (over 500 trucks/day). The surface layer thicknesses considered ranged from 6 inches to 14 inches for JPCP and 0 to 8 inches for HMA layers. Results showed that project-level calibrations reduced default model prediction error by nearly twice that of network-level calibration. Table A.7 and Table A.8, as reported from this study, contain coefficients for the smoothness model of HMA overlays of rigid pavements and JPCP.

Table A.7. HMA overlaid rigid pavements' IRI calibration coefficients for surface layer thickness within ADTT (Schram and Abdelrahman 2006)

ADTT	Thickness	C1	C2	C3	N	R^2	SEE (m/km)
Low	2"-3"	0.1318	0.0018	0.3971	3	0.994	0.02
	4"-5"	0.0704	-0.0048	-2.8771	16	0.813	0.11
	5"-6"	-0.0038	0.2409	-4.6360	5	0.039	1.15
Medium	2"-3"	0.0639	0.1337	-0.7896	21	0.612	0.5
	3"-4"	0.0733	0.0282	1.4725	65	0.532	0.36
	4"-5"	0.0781	-0.0032	1.1116	82	0.546	0.31
	5"-6"	0.0649	0.0169	3.5543	84	0.535	0.31
	6"-7"	0.0794	-0.0312	4.3652	31	0.888	0.17
	7"-8"	0.0674	-0.0164	1.7122	19	0.674	0.13
	8"-9"	0.0683	0.0192	-3.6231	13	0.936	0.1
High	0"-1"	0.2019	0.1158	-10.0646	27	0.392	0.45
	2"-3"	0.1866	0.0498	-16.7082	19	0.565	0.6
	3"-4"	0.1835	-0.0579	8.1863	32	0.010	0.9
	4"-5"	0.1170	-0.0100	1.4057	101	0.299	0.51
	5"-6"	0.2422	0.0371	-23.4448	62	0.713	0.85
	6"-7"	0.0756	0.0127	0.9250	64	0.597	0.22
	7"-8"	0.0604	0.0574	-2.4936	7	0.624	0.2
	8"-9"	0.0578	0.0706	-10.9179	28	0.103	0.25
	9"-10"	0.1005	-0.0001	-0.5216	8	0.845	0.13

Table A.8. JPCP IRI calibration coefficients for surface layer thickness within ADTT (Schram and Abdelrahman 2006)

ADTT	Thickness	C1	C2	C3	C4	N	R^2	SEE (in/mi)
Low	6"-7"	0.0000	0.0000	1.0621	74.8461	33	0.434	26.885
	7"-8"	0.0000	0.0000	1.9923	46.9256	37	0.961	8.235
	8"-9"	0.8274	0.0000	0.0000	86.9721	39	0.904	14.465
	9"-10"	0.3458	0.0000	1.5983	64.3453	110	0.537	26.230
	10"-11"	0.0300	0.0000	3.4462	10.7893	37	0.893	17.280
	11"-12"	--	--	--	--	--	--	--
	12"-13"	--	--	--	--	--	--	--
	13"-14"	--	--	--	--	--	--	--
	14"-15"	--	--	--	--	--	--	--
Medium	6"-7"	0.0000	0.0000	4.1422	0.0000	3	0.966	5.094
	7"-8"	0.0000	1.5628	0.0000	71.9009	22	0.968	9.952
	8"-9"	0.0000	0.0000	1.7162	53.0179	122	0.291	40.537
	9"-10"	0.1910	0.0000	0.9644	89.3990	609	0.686	24.945
	10"-11"	0.0000	0.0000	2.0945	73.1246	314	0.812	18.535
	11"-12"	0.0000	0.0090	1.3617	100.0000	27	0.792	10.166
	12"-13"	--	--	--	--	--	--	--
	13"-14"	0.0000	0.0100	2.2226	24.9354	4	0.924	3.948
	14"-15"	--	--	--	--	--	--	--
High	6"-7"	--	--	--	--	--	--	--
	7"-8"	--	--	--	--	--	--	--
	8"-9"	0.0000	0.1376	0.4352	79.5526	46	0.151	48.576
	9"-10"	0.1561	0.0000	1.1024	62.9556	81	0.333	31.255
	10"-11"	0.0000	0.0000	1.6344	100.0000	228	0.653	22.295
	11"-12"	0.1125	1.8207	1.1678	100.0000	29	0.739	13.366
	12"-13"	0.0000	0.0000	1.5331	100.0000	151	0.719	17.724
	13"-14"	0.0100	0.0100	0.5184	0.0000	4	0.623	1.728
	14"-15"	0.1904	0.0000	2.1387	51.4053	146	0.838	9.018

Muthadi and Kim (2008) performed the calibration of MEPDG for flexible pavements located in North Carolina (NC) using version 1.0 of MEPDG software. Two distress models, rutting and alligator cracking, were used for this effort. A total of 53 pavement sections were selected from the LTPP program and the NC DOT databases for the calibration and validation process. Based on calibration procedures suggested by NCHRP 1-40B study, the flow chart was made for this study. The verification results of MEPDG performance models with national calibration factors showed bias (systematic difference) between the measured and predicted distress values. The Microsoft Excel Solver program was used to minimize the sum of the squared errors (SSE) of the measured and the predicted rutting or cracking by varying the coefficient parameters of the transfer function. Table A.9 lists local calibration factors of rutting and alligator cracking transfer functions obtained in this study. This study concluded that the standard error for the rutting model and the alligator cracking model is significantly less after the calibration.

Table A.9. North Carolina local calibration factors of rutting and alligator cracking transfer functions (Muthadi and Kim 2008)

Recalibration	Calibration Coefficient	National Calibration	National Recalibration	Local Calibration
Rutting				
AC	k_1	−3.4488	−3.35412	−3.41273
	k_2	1.5606	1.5606	1.5606
	k_3	0.479244	0.479244	0.479244
GB	β_{GB}	1.673	2.03	1.5803
SG	β_{SG}	1.35	1.67	1.10491
Fatigue				
AC	k_1	0.00432	0.007566	0.007566
	k_2	3.9492	3.9492	3.9492
	k_3	1.281	1.281	1.281
	C_1	1	1	0.437199
	C_2	1	1	0.150494

The Washington State DOT (Li et al. 2009) developed procedures to calibrate the MEPDG (version 1.0) flexible pavement performance models using data obtained from the Washington State Pavement Management System (WSPMS). Calibration efforts were concentrated on the asphalt mixture fatigue damage, longitudinal cracking, alligator cracking, and rutting models. There were 13 calibration factors to be considered in the four related models. An elasticity analysis was conducted to describe the effects of those calibration factors on the pavement distress models. I.e., the higher the absolute value of elasticity, the greater impact the factor has on the model. The calibration results of typical Washington State flexible pavement systems determined from this study presents in Table A.10. This study also reported that a version 1.0 of MEPDG software bug does not allow calibration of the roughness model.

Table A.10. Local calibrated coefficient results of typical Washington State flexible pavement systems (Li et al. 2009)

Calibration Factor		Default	Calibrated Factors
AC Fatigue	B_{f1}	1	0.96
	B_{f2}	1	0.97
	B_{f3}	1	1.03
Longitudinal cracking	C1	7	6.42
	C2	3.5	3.596
	C3	0	0
	C4	1000	1000
Aligator cracking	C1	1	1.071
	C2	1	1
	C3	6000	6000
AC Rutting	B_{r1}	1	1.05
	B_{r2}	1	1.109
	B_{r3}	1	1.1
Subgrade Rutting	B_{s1}	1	0
IRI	C1	40	–
	C2	0.4	–
	C3	0.008	–
	C4	0.015	–

Similar to the study conducted in NC (Muthadi and Kim 2008), Banerjee et al. (2009) minimized the SSE between the observed and the predicted surface permanent deformation to determine the coefficient parameters (β_{r1} and β_{r3}) of HMA permanent deformation performance model after values based on expert knowledge assumed for the subgrade permanent deformation calibration factors (β_{s1}) and the HMA mixture temperature dependency calibration factors (β_{r2}). Pavement data from the Texas SPS-1 and SPS-3 experiments of the LTPP database were used to run the MEPDG and calibrate the guide to Texas conditions. The set of state-default calibration coefficients for Texas was determined from joint minimization of the SSE for all the sections after the determination of the Level 2 input calibration coefficients for each section. The results of calibration factors as obtained from this study are given in Figure A.9. Banerjee et al. (2011) also determined the coefficient parameters (β_{r1} *and* β_{r3}) of rutting for rehabilitated flexible pavements under six of regional area in U.S.

Souliman et al. (2010) presented the calibration of the MEPDG (Version 1.0) predictive models for flexible pavement design in Arizona conditions. This calibration was performed using 39 Arizona pavement sections included in the LTPP database. The results of calibration factors as obtained from this study are given in Table A.11.

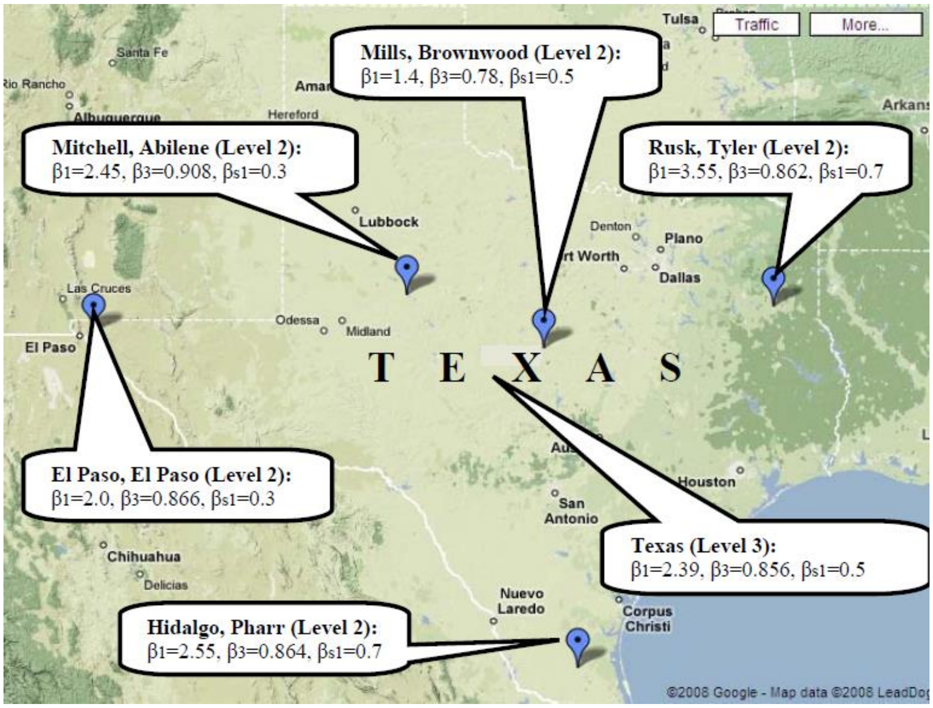


Figure A.9. Regional and state level calibration coefficients of HMA rutting depth transfer function for Texas (Banerjee et al. 2009)

Table A.11. Calibration coefficients of the MEPDG flexible pavement distress models in Arizona conditions (Souliman et al. 2010)

MEPDG Model	Coefficients before Calibration	Coefficients after Calibration	Net Effect of Calibration
Alligator Fatigue Transfer Function	$\beta_{f1} = 1$	$\beta_{f1} = 0.729$	Increased prediction
	$\beta_{f2} = 1$	$\beta_{f2} = 0.8$	
	$\beta_{f3} = 1$	$\beta_{f3} = 0.8$	
	$C_1 = 1.0$	$C_1 = 0.732$	
	$C_2 = 1.0$	$C_2 = 0.732$	
Longitudinal Fatigue Transfer Function	$\beta_{f1} = 1$	$\beta_{f1} = 0.729$	Decreased prediction
	$\beta_{f2} = 1$	$\beta_{f2} = 0.8$	
	$\beta_{f3} = 1$	$\beta_{f3} = 0.8$	
	$C_1 = 7.0$	$C_1 = 1.607$	
	$C_2 = 3.5$	$C_2 = 0.803$	
AC Rutting Model	$\beta_{r1} = 1$	$\beta_{r1} = 3.63$	Increased prediction
	$\beta_{r2} = 1$	$\beta_{r2} = 1.1$	
	$\beta_{r3} = 1$	$\beta_{r3} = 0.7$	
Granular base Rutting Model	$\beta_{gb} = 1$	$\beta_{gb} = 0.111$	Decreased prediction
Subgrade Rutting Model	$\beta_{sg} = 1$	$\beta_{sg} = 1.38$	Increased prediction
Roughness Model	$C_1 = 40$	$C_1 = 5.455$	Decreased prediction
	$C_2 = 0.4$	$C_2 = 0.354$	
	$C_3 = 0.008$	$C_3 = 0.008$	
	$C_4 = 0.015$	$C_4 = 0.015$	

Hoegh et al. (2010) utilized time history rutting performance data for pavement sections at the Minnesota Department of Transportation (MnDOT) full-scale pavement research facility (MnROAD) for an evaluation and local calibration of the MEPDG rutting model. Instead of an adjustment of the calibration parameters in current MEPDG rutting model, a modified rutting model was suggested to account for the forensic and predictive evaluations on the local conditions. This study demonstrated that current MEPDG subgrade and base rutting models grossly overestimate rutting for the MnROAD test sections. Instead of calibration of fatigue cracking performance model, Velasquez et al (2009) calibrated MEPDG fatigue damage model against MnPAVE which is mechanistic-empirical design based software calibrated in Minnesota. The alligator cracking predicted by the MEPDG was approximately 5 times greater than that predicted by MnPAVE. This difference has been minimized by setting up 0.1903 of fatigue damage model coefficient B_{f1}.

Glover and Mallela (2009) calibrated MEPDG rutting and IRI models by using LTPP data of Ohio roads. Due to lack data (no distress observation or record), the other distress predictions were not calibrated. Similar to Ohio study, Darter et al (2009) could calibrate only MEPDG rutting model due to lack of data. However, they found the national calibrated IRI model of flexible pavement produce good of fit between measured and prediction IRI and SEE approximately the same as that reported in NCHRP 1-37A study.

Some type of maintenance or rehabilitation activity can make actual distress measurements decrease in distress time-history plots (Kim et al 2010). Banerjee et al. (2010) found that the calculation factors of MEPDG permanent deformation performance models are influenced by maintenance strategies. Liu et al. (2010) suggested historical pavement performance model to account for rehabilitation or maintenance activity using piecewise approximation. The whole pavement serviceable life was divided into three zones: Zone1 for the early age pavement distress, Zone 2 in rehabilitation stage, and Zone 3 for over-distressed situations. The historical pavement performance data were regressed independently in each time zone. This approach is able to accurately predict the pavement distress progression trends in each individual zone by eliminating the possible impacts from the biased data in the other zones. It is also possible to compare the pavement distress progression trends in each individual zone with the MEPDG incremental damage approach predictions.

Mamlouk and Zapata (2010) discussed differences between the Arizona Department of Transportation (ADOT) PMS data and the LTPP database used in the original development and national calibration of the MEPDG distress models. Differences were found between: rut measurements, asphalt cracking, IRI, and all layer backcalculated moduli found from NDT measurements done by ADOT and those of the LTPP. Differences in distress data include types of data measured, types of measuring equipment, data processing methods, units of measurements, sampling methods, unit length of pavement section, number of runs of measuring devices, and survey manuals used. Similar findings were reported in NC DOT PMS by Corley-Lay et al. (2010).

Hall et al (2011) also discussed differences in defining transverse cracking between the MEPDG and LTPP distress survey manual. The transverse cracking in MPEG is related to thermal cracking caused by thermal stress in pavement while one in LTPP distress survey manual is the cracks predominately perpendicular to pavement centerline by various causes. Since the pavement sections selected in this study are generally in good condition for transverse cracking and rutting, local calibration coefficients were optimized for the alligator cracking and longitudinal cracking.

Jadoun (2011) recalibrated rutting and top down cracking models with transfer functions for North Carolina flexible pavements using two optimization approaches of a generalized reduced gradient (GRG) method and a genetic algorithm (GA). The subgrade material properties required were extracted from the NCHRP 9-23A national soils database (Zapata 2010) by using a GIS-based methodology. Regarding traffic characterization required, the sensitive analysis were conducted to identify traffic inputs having significant effect on performance predictions. The values of sensitive traffic inputs are clustered into groups having similarity by using 48 hour weight motion data.

Rigid Pavements

The Washington State DOT (Li et al. 2006) developed procedures to calibrate the MEPDG (Version 0.9) rigid pavement performance models using data obtained from the WS PMS. Some significant conclusions from this study are as follows: (a) WSDOT rigid pavement performance prediction models require calibration factors significantly different from default values; (b) the MEPDG software does not model longitudinal cracking of rigid pavement, which is significant in WSDOT pavements; (c) WS PMS does not separate longitudinal and transverse cracking in rigid pavements, a deficiency that makes calibration of the software's transverse cracking model difficult; and (d) the software does not model studded tire wear, which is significant in WS DOT pavements. This study also reported that: (a) the calibrated software can be used to predict future deterioration caused by faulting, but it cannot be used to predict cracking caused by the transverse or longitudinal cracking issues in rigid pavement, and (b) with a few improvements and resolving software bugs, MEPDG software can be used as an advanced tool to design rigid pavements and predict future pavement performance. The local calibration results of typical Washington State rigid pavement systems determined from this study are presented in Table A.12.

Table A.12. Calibration coefficients of the MEPDG (Version 0.9) rigid pavement distress models in the State of Washington (Li et al. 2006)

Calibration Factor		Default for New Pavements	Undoweled	Undoweled – MP[a]	DBR[b,c]
Cracking	C_1	2	2.4	2.4	2.4
	C_2	1.22	1.45	1.45	1.45
	C_4	1	0.13855	0.13855	0.13855
	C_5	-1.68	-2.115	-2.115	-2.115
Faulting	C_1	1.29	0.4	0.4	0.934
	C_2	1.1	0.341	0.341	0.6
	C_3	0.001725	0.000535	0.000535	0.001725
	C_4	0.0008	0.000248	0.000248	0.0004
	C_5	250	77.5	77.5	250
	C_6	0.4	0.0064	0.064	0.4
	C_7	1.2	2.04	9.67	0.65
	C_8	400	400	400	400
Roughness[d]	C_1	0.8203	0.8203	0.8203	0.8203
	C_2	0.4417	0.4417	0.4417	0.4417
	C_3	1.4929	1.4929	1.4929	1.4929
	C_4	25.24	25.24	25.24	25.24

Notes:

a. Mountain pass climate
b. Dowel bar retrofitted
c. DBR calibration factors are the same as default "restoration" values in NCHRP 1-37A software
d. Roughness calibration factors are the same as the default values

Khazanovich et al. (2008) evaluated MEPDG rigid pavement performance prediction models for the design of low-volume concrete pavements in Minnesota. It was found that the faulting model in MEPDG version 0.8 and 0.9 produced acceptable predictions, whereas the cracking model had to be adjusted. The cracking model was recalibrated using the design and performance data for 65 pavement sections located in Minnesota, Iowa, Wisconsin, and Illinois. The recalibrated coefficients of MEPDG 0.8 and 0.9 cracking model predictions in this study are (1) $C_1 = 1.9875$, (2) $C_2 = -2.145$. These values are recalibrated into $C_1 = 0.9$ and $C_2 = -2.64$ by using the MEPDG version 1.0 (Velasquez et al 2009). Since MEPDG software evaluated in these studies was not a final product, authors recommended that these values should be updated for the final version of the MEPDG software.

Darter et al. (2009) found that the national calibrated MEPDG model predicted faulting, transverse cracking and IRI well under Utah conditions with an adequate goodness of fit and no significant bias. Bustos et al. (2009) attempted to adjust and calibrate the MEPDG rigid pavement distress models in Argentina conditions. A sensitivity analysis of distress model transfer functions was conducted to identify the most important calibration coefficient. The C_6 of joint faulting model transfer function and the C_1 or C_2 of cracking model transfer function were the most sensitive coefficients. Delgadillo et al (2011) also present local calibration coefficients of transverse cracking and faulting of JPCP in Chile.

APPENDIX B. SCREEN SHOTS OF CALIBRATION TOOL SECTIONS IN MEPDG SOFTWARE (VERSION 1.1)

New Rigid Pavement

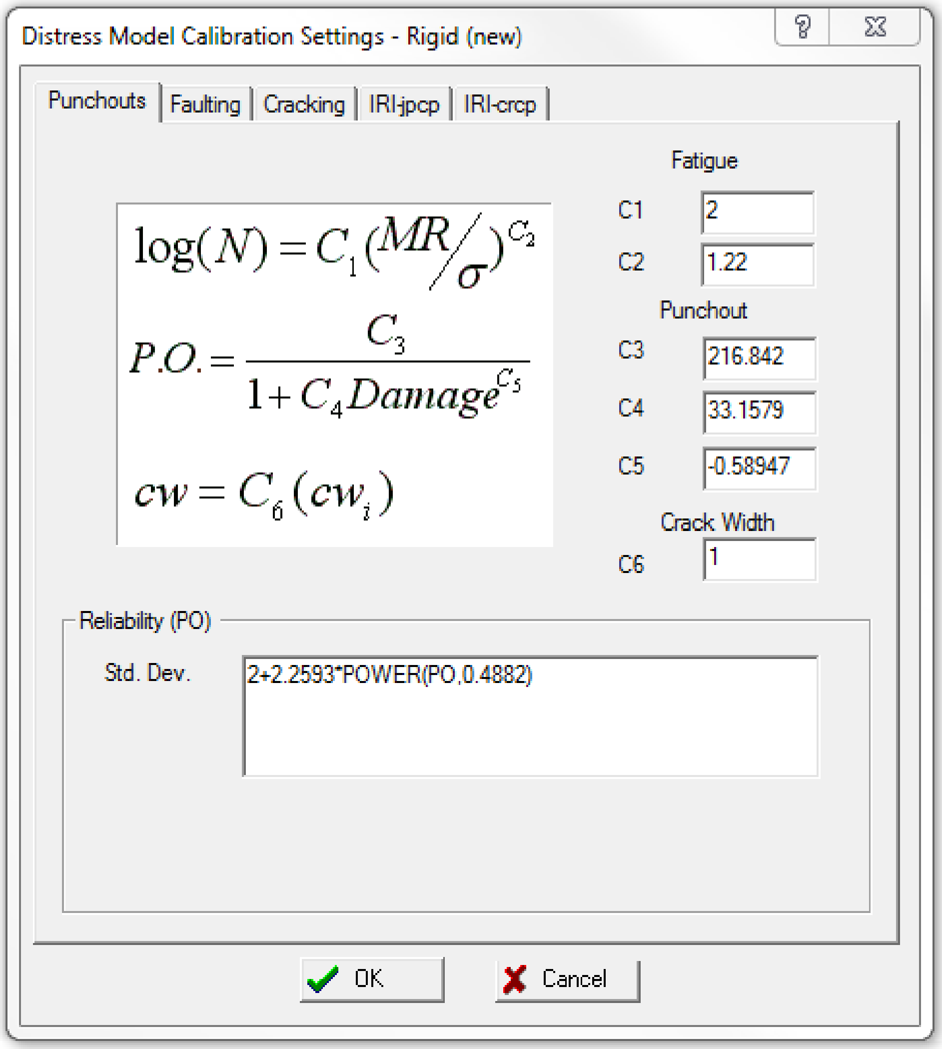


Figure B.1. Punchout of new PCC pavements

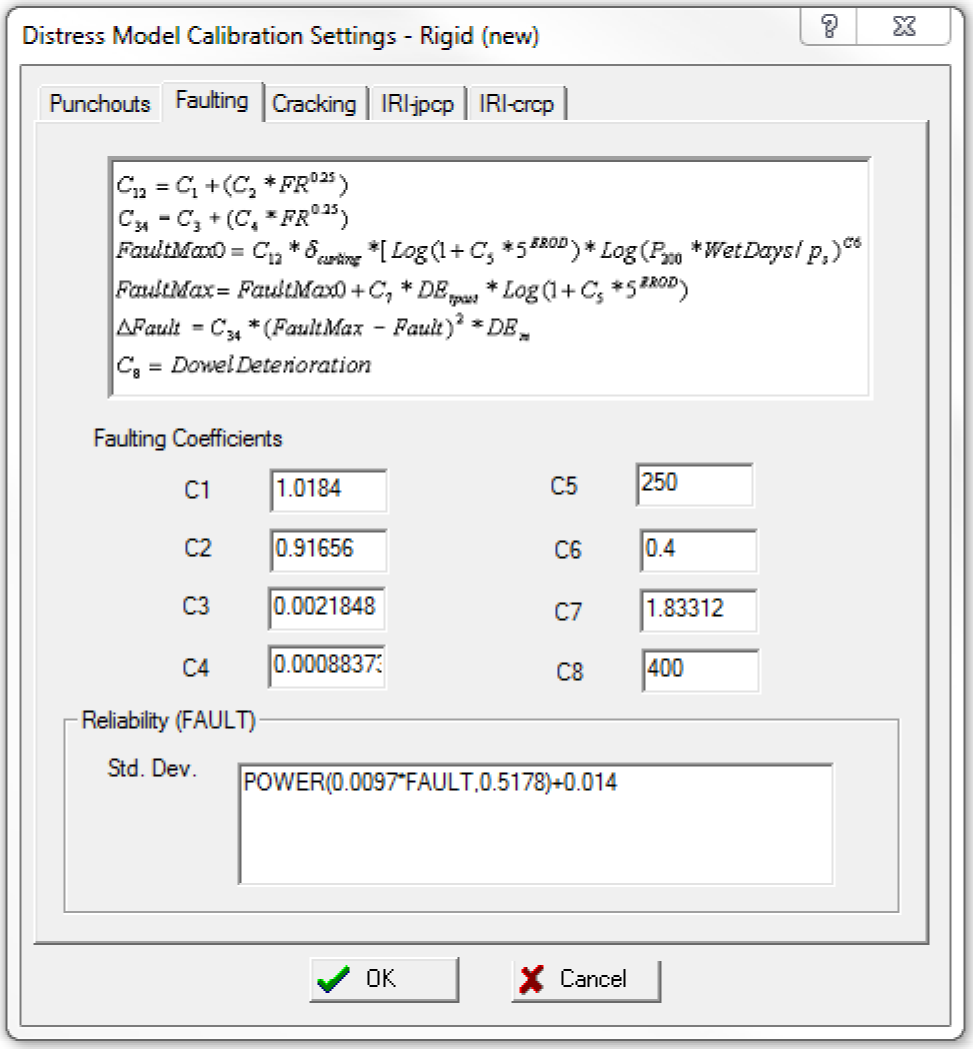


Figure B.2. Faulting of new PCC pavements

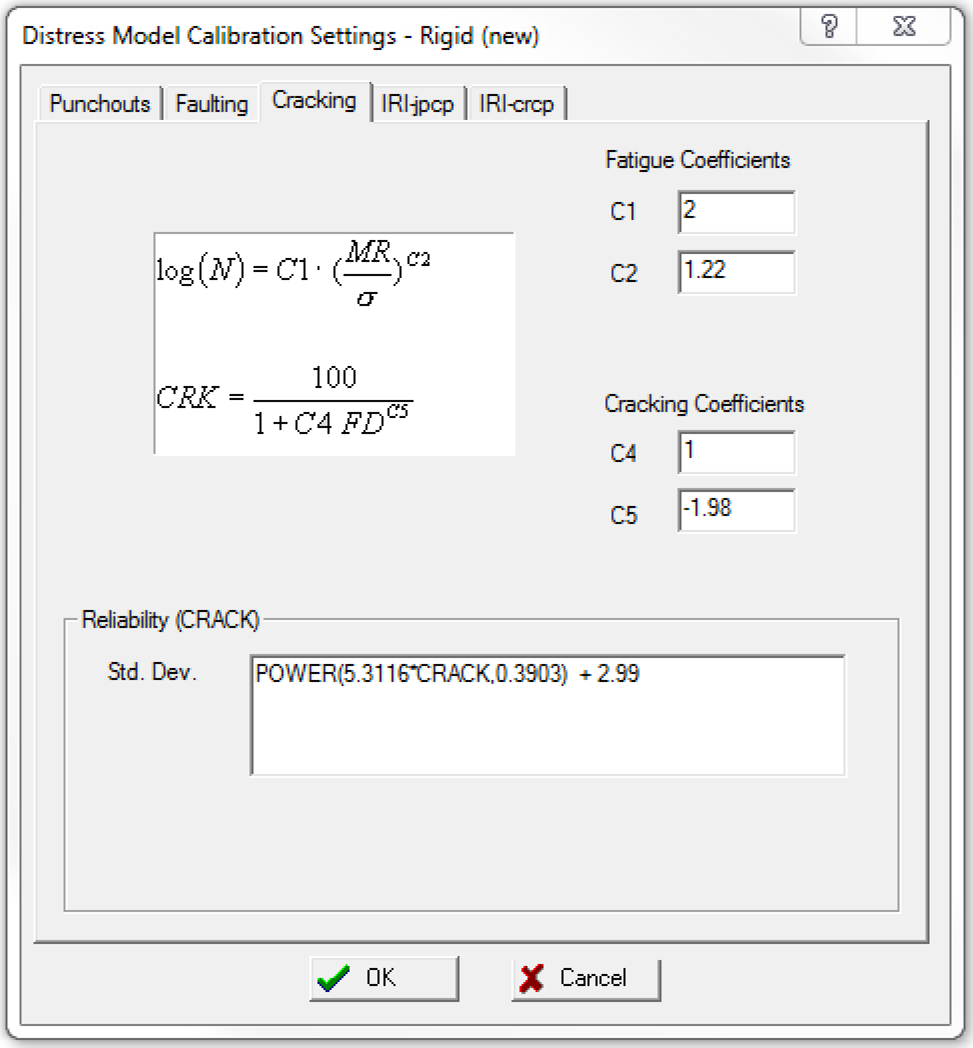


Figure B.3. Cracking of new PCC pavements

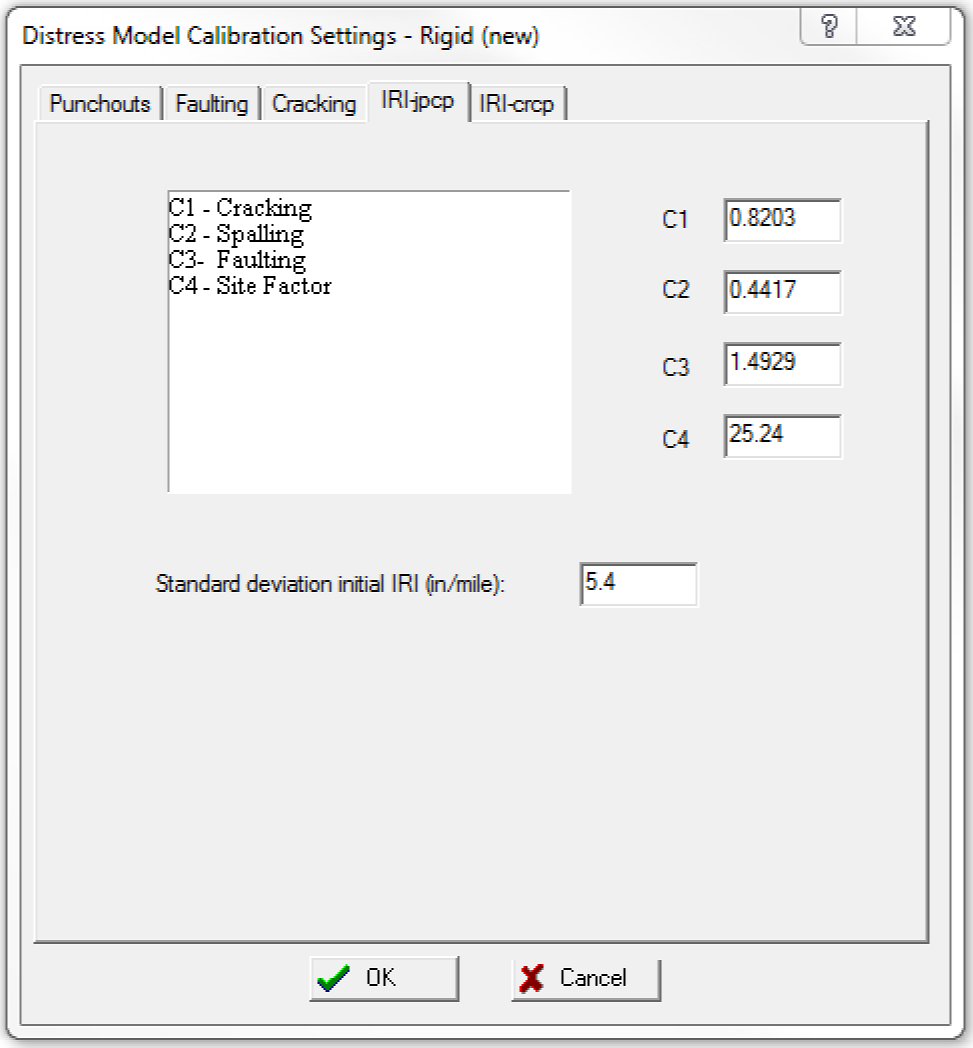


Figure B.4. IRI - JPCP of new PCC pavements

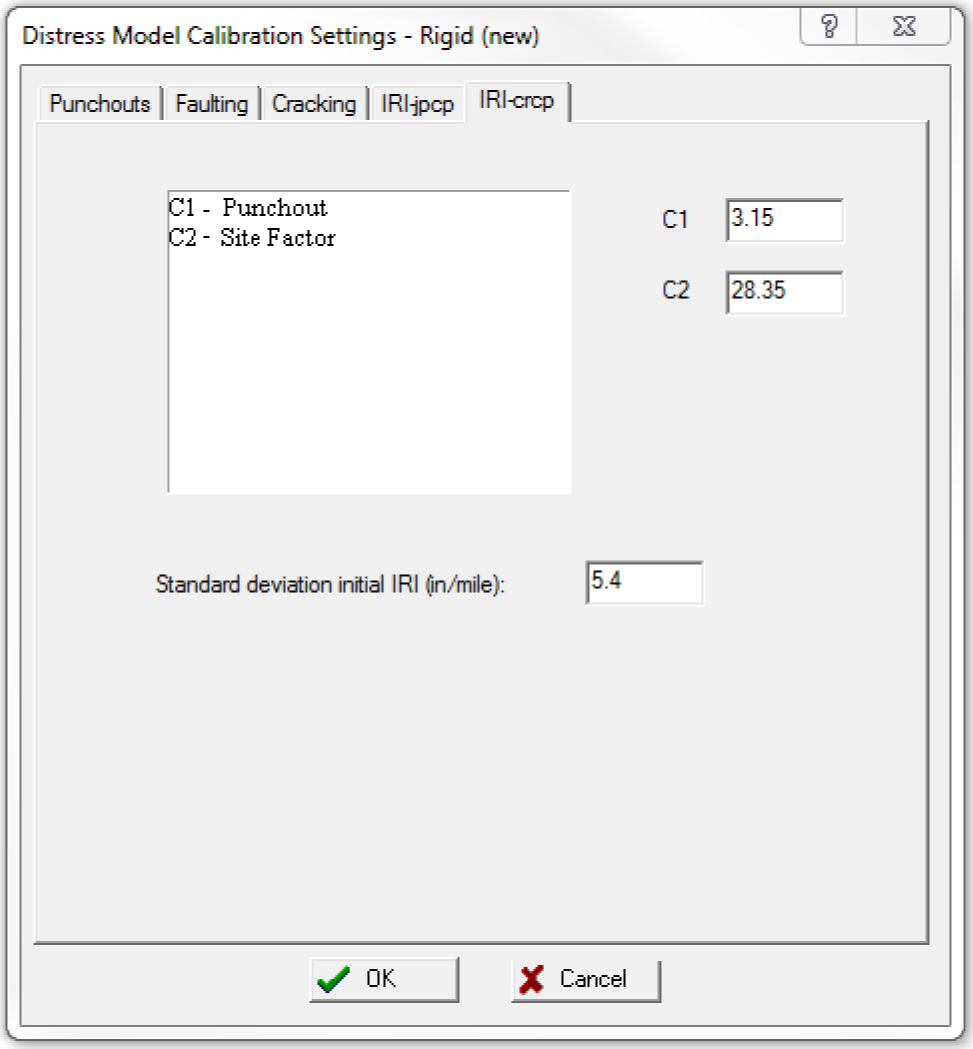


Figure B.5. IRI - CRCP of new PCC pavements

New Flexible Pavement

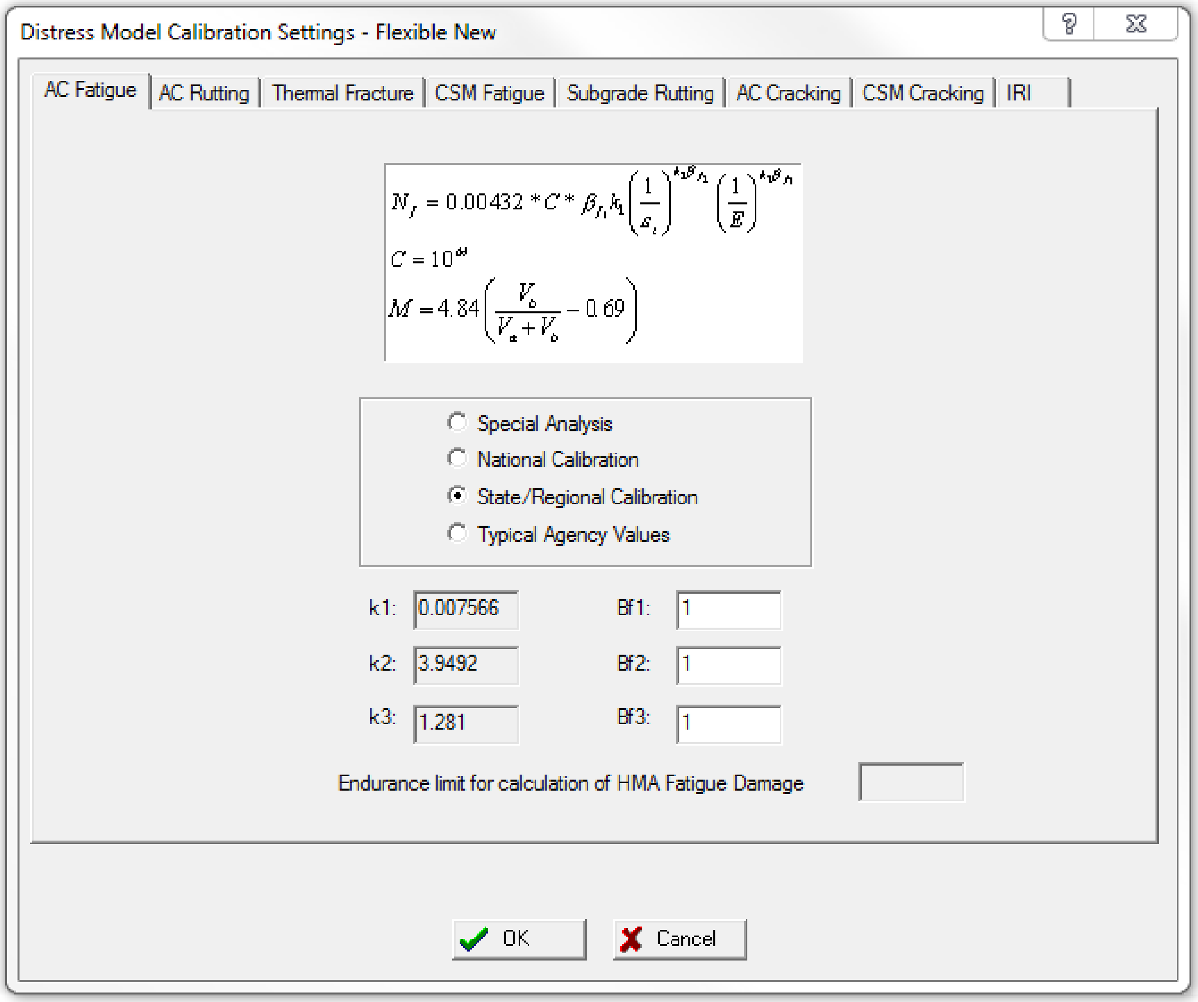


Figure B.6. AC fatigue of HMA pavements

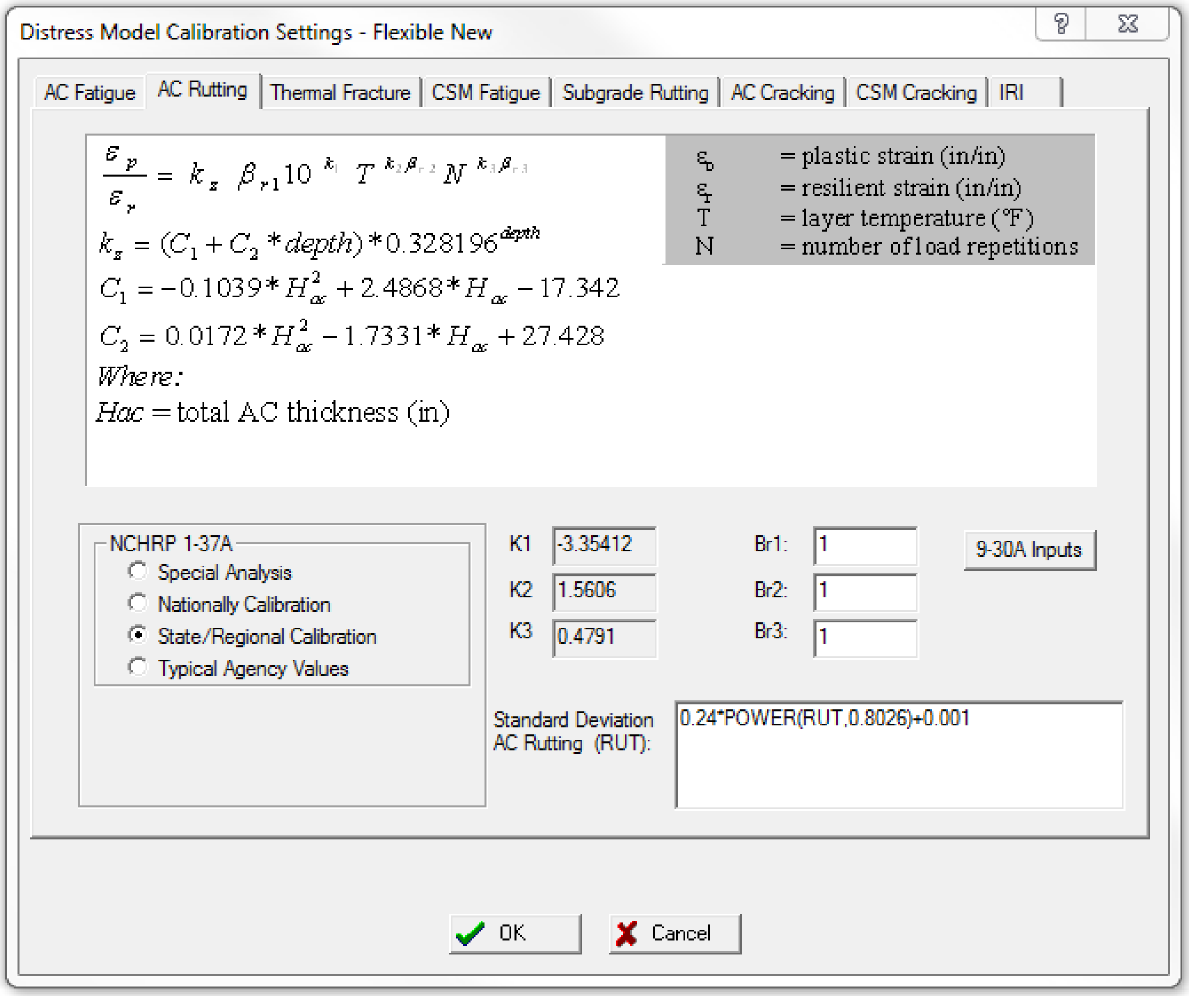


Figure B.7. AC rutting of HMA pavements

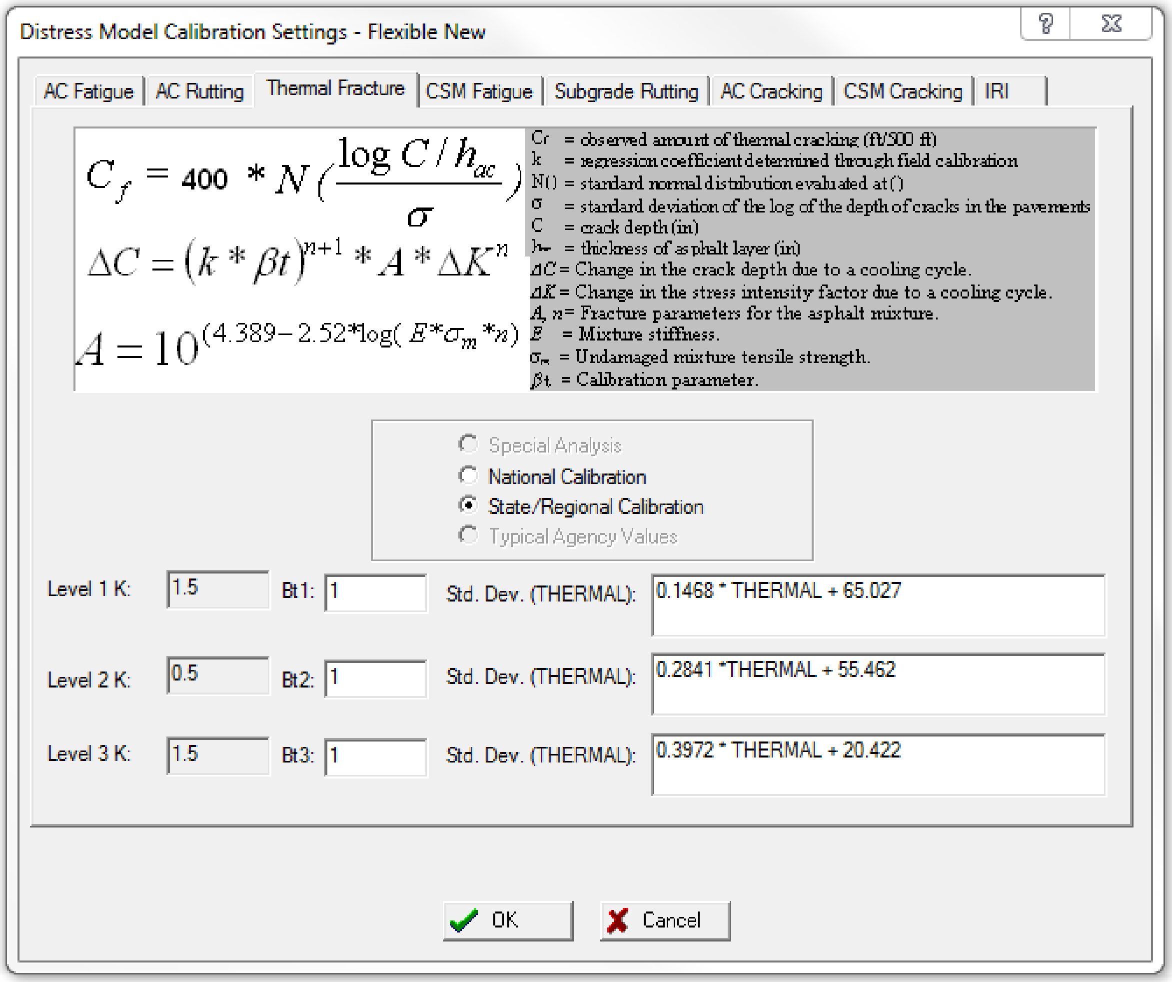


Figure B.8. Thermal fracture of HMA pavements

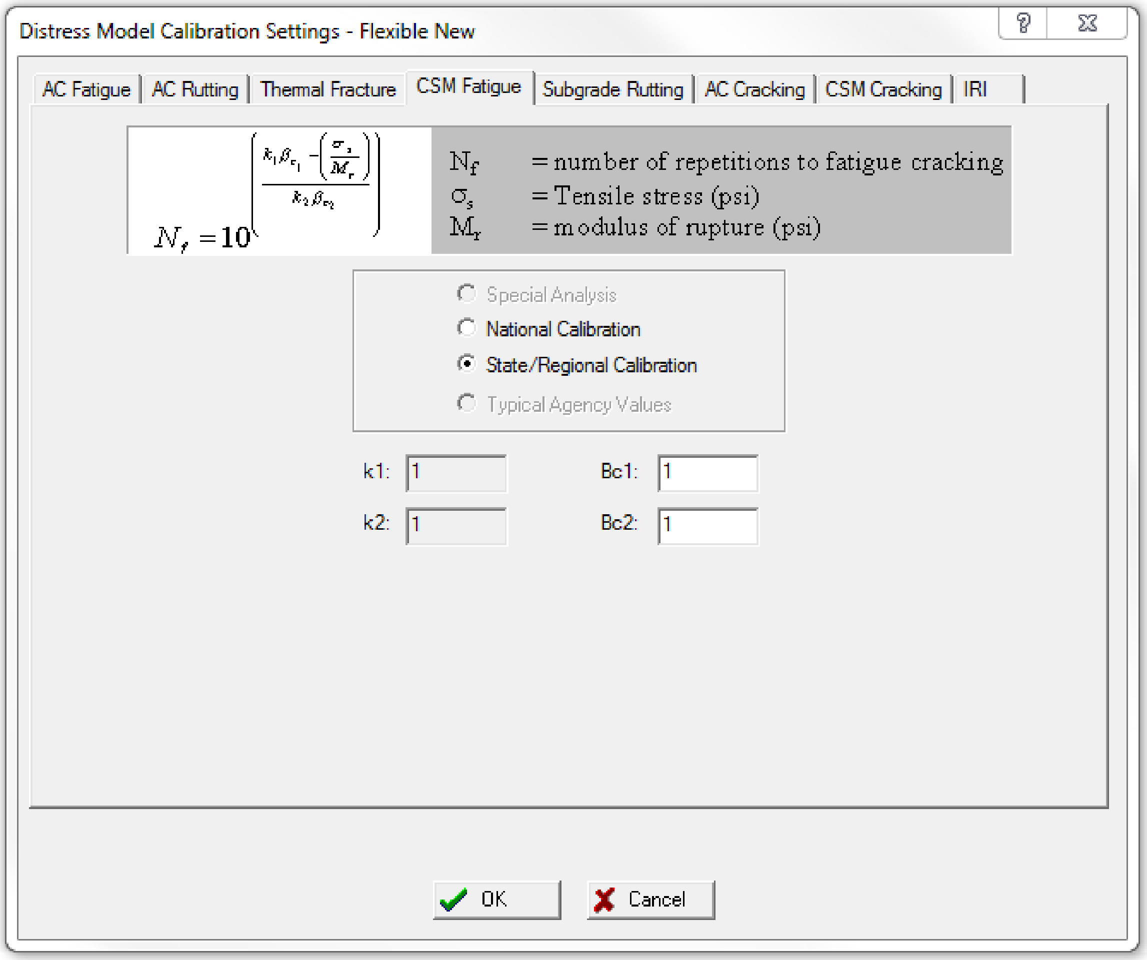


Figure B.9. CSM fatigue of HMA pavements

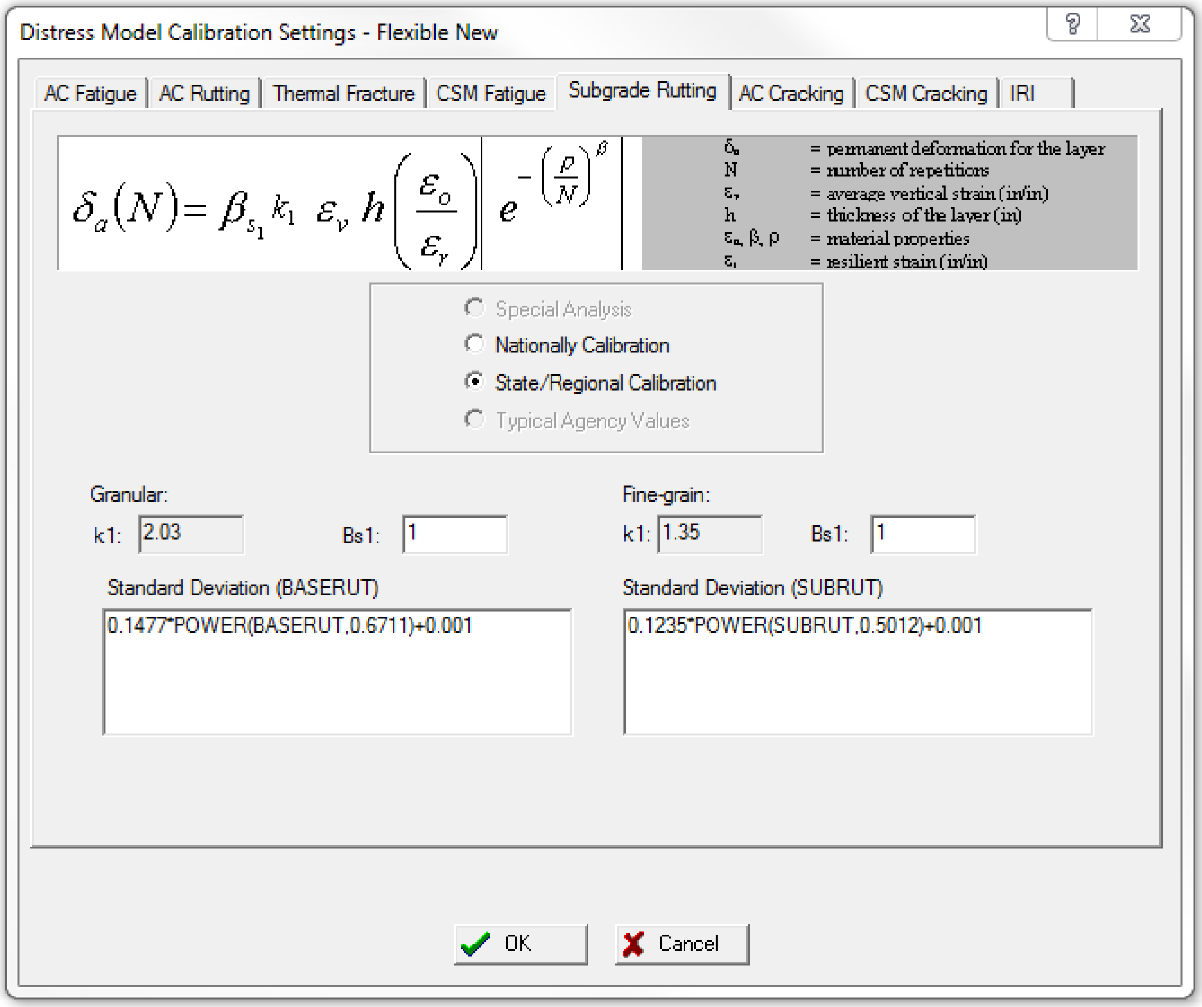


Figure B.10. Subgrade rutting of HMA pavements

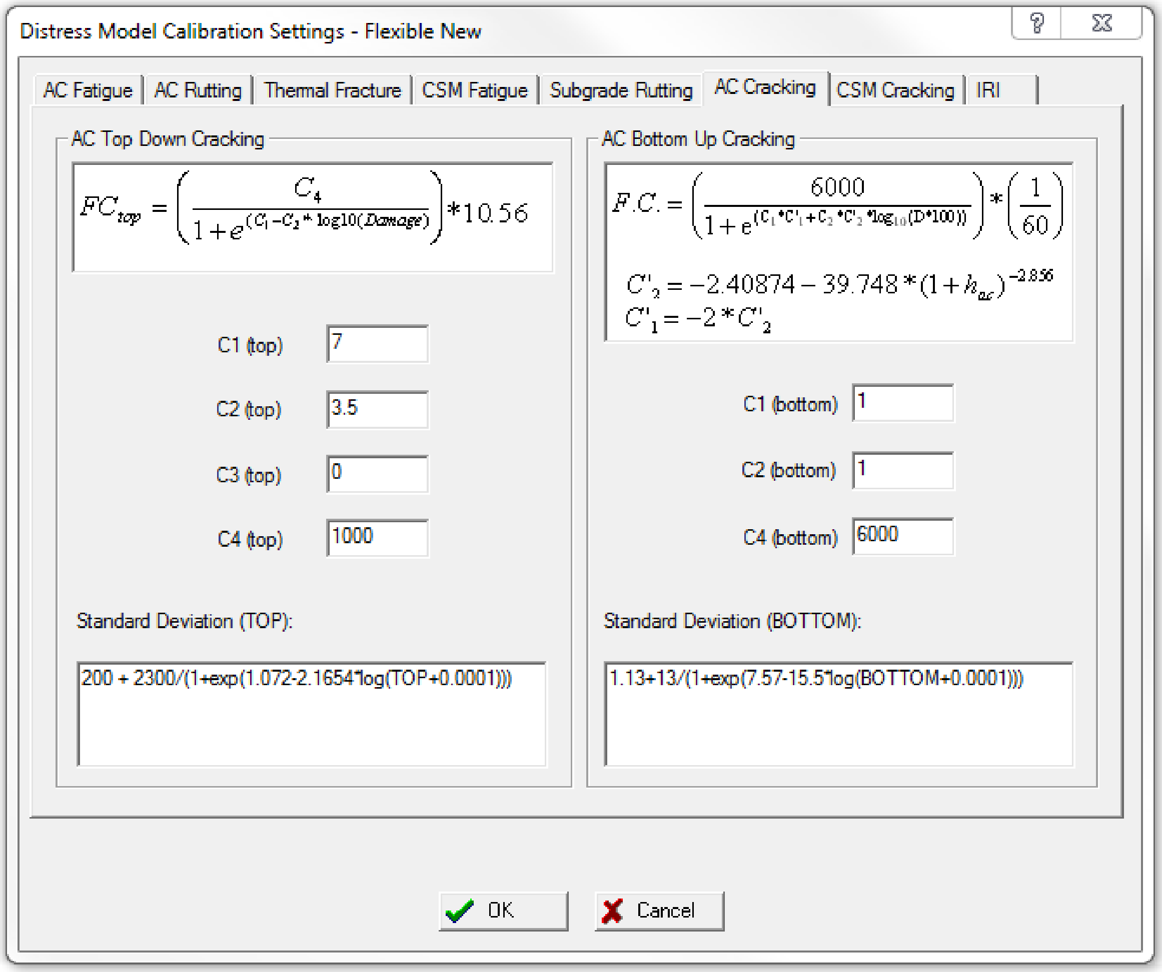


Figure B.11. AC cracking of HMA pavements

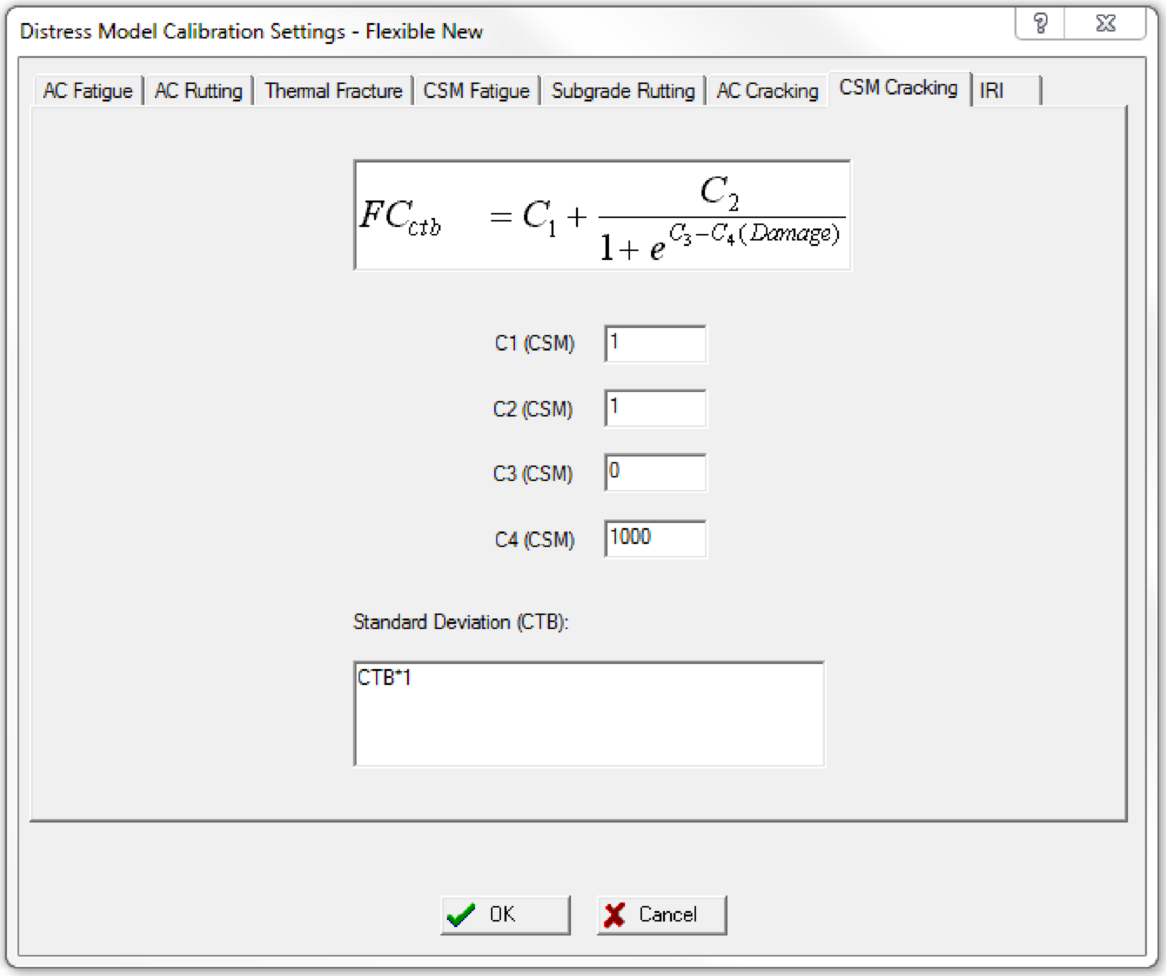


Figure B.12. CSM cracking of HMA pavements

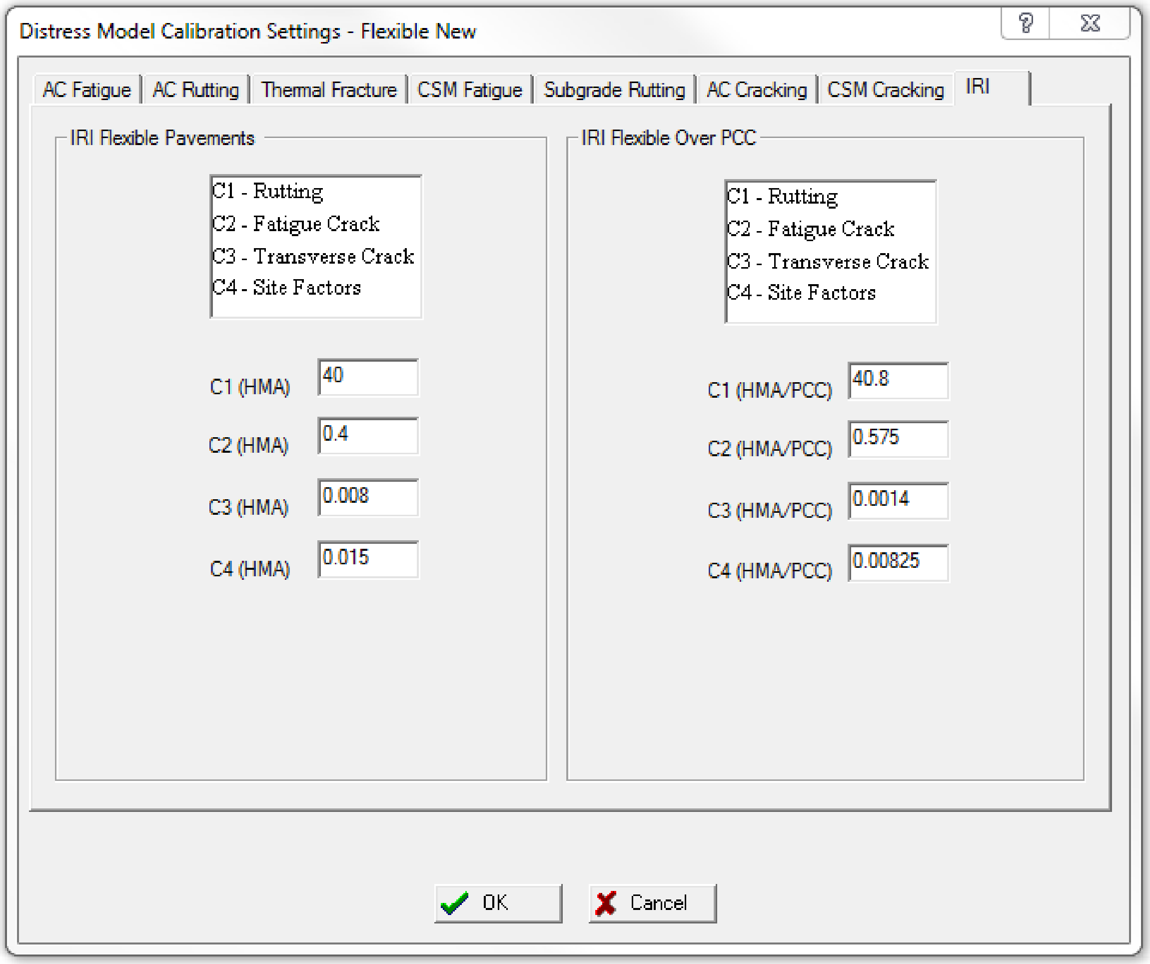


Figure B.13. IRI of HMA pavements

Rehabilitated Flexible Pavement

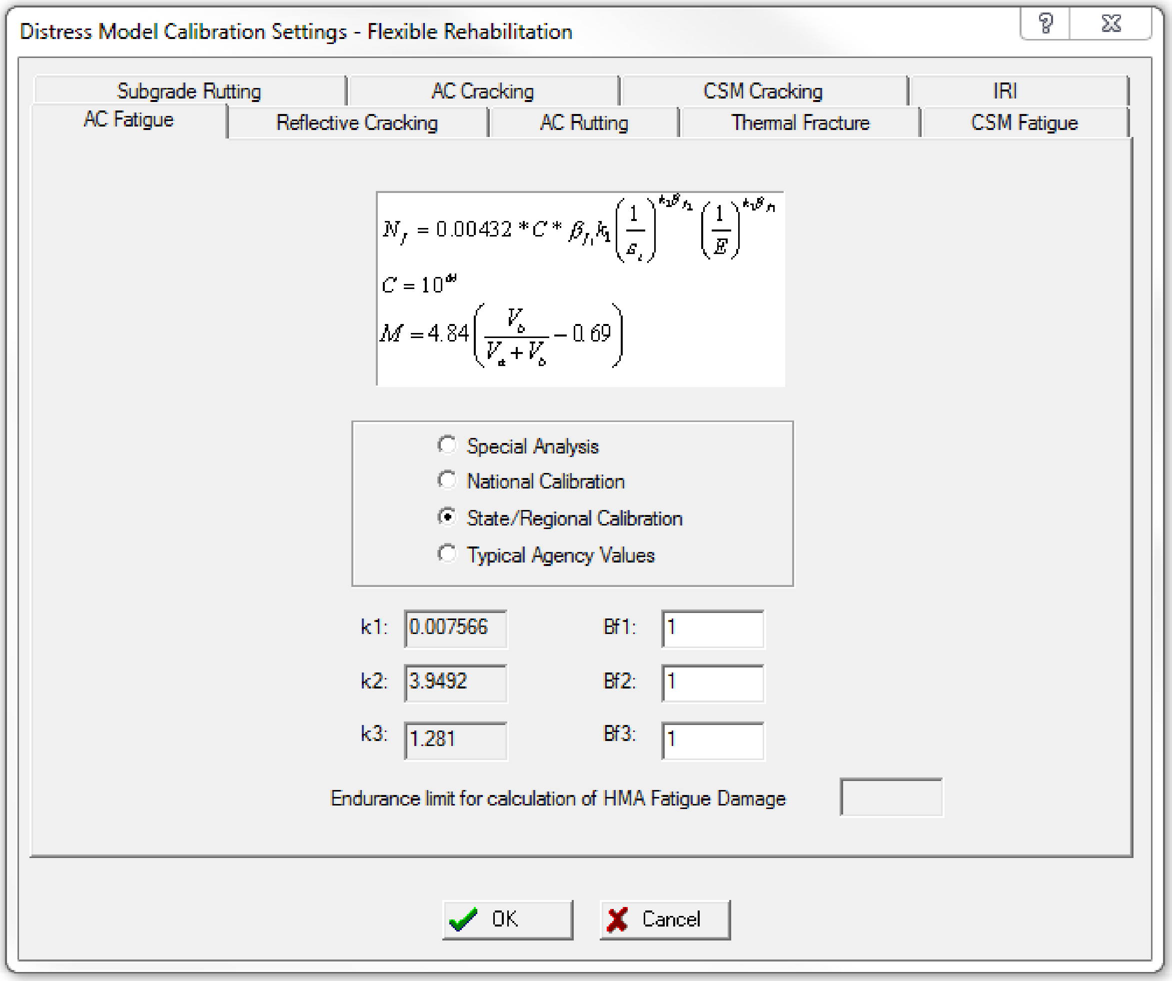


Figure B.14. AC fatigue of rehabilitated HMA pavements

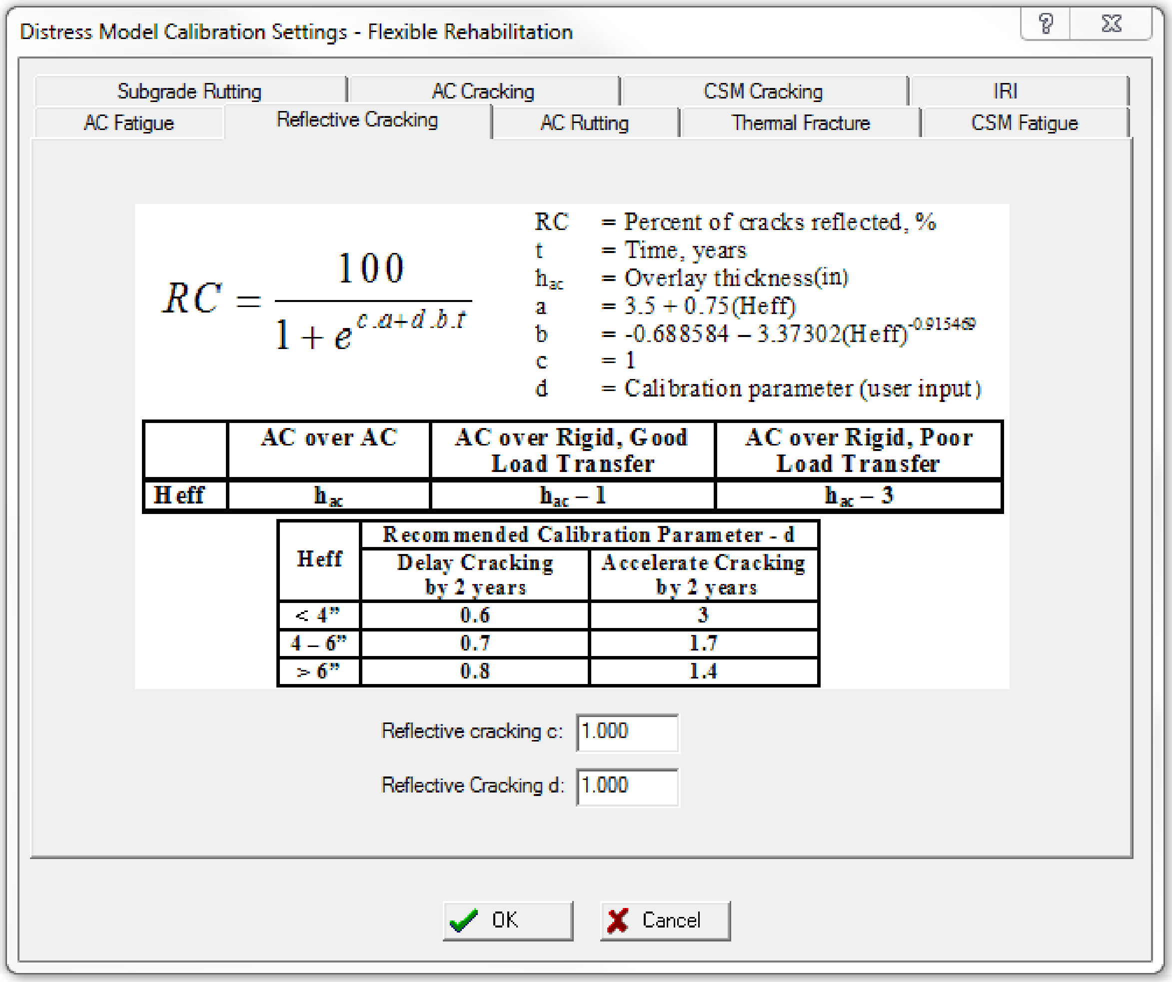


Figure B.15. Reflective cracking of rehabilitated HMA pavements

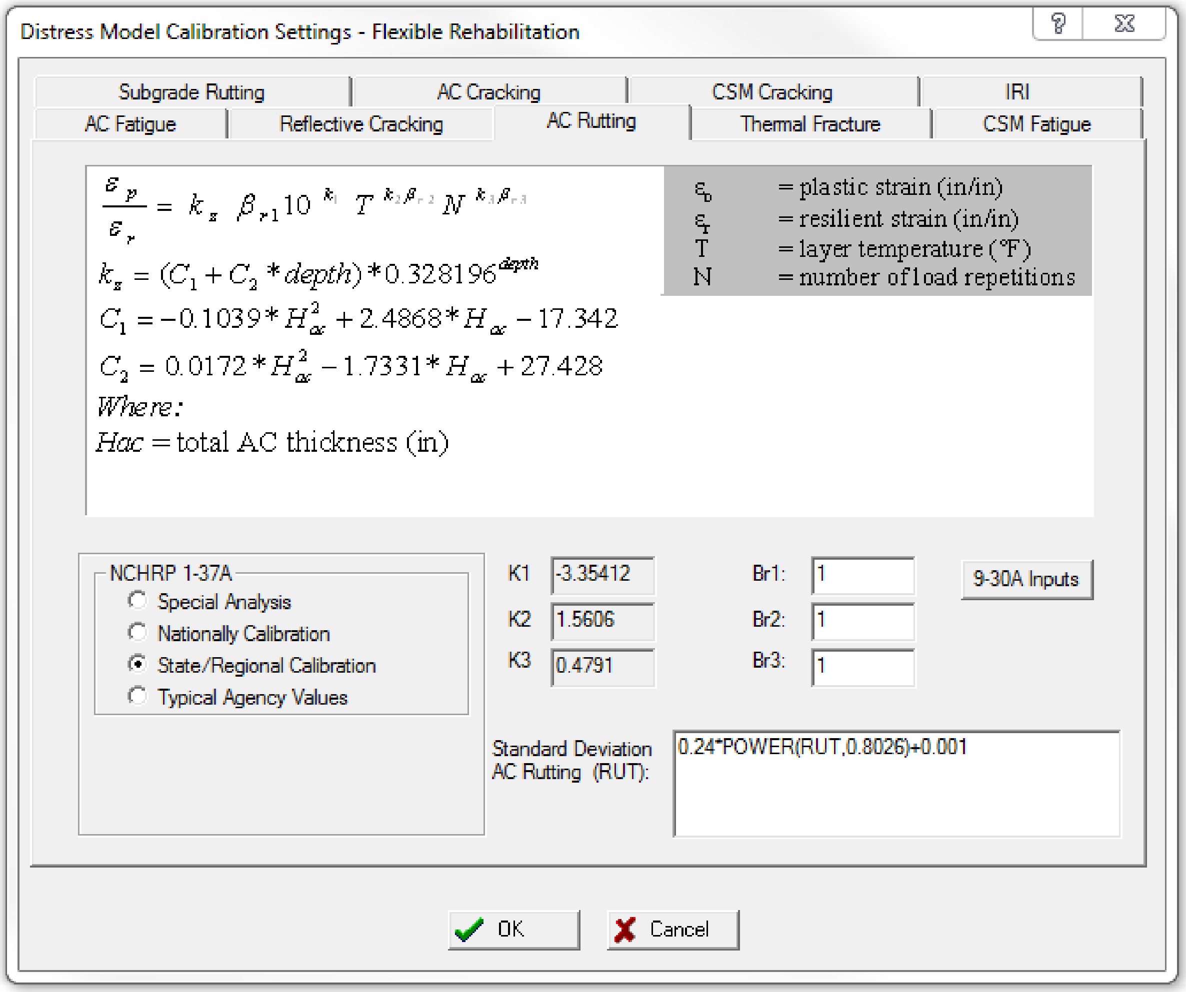


Figure B.16. AC rutting of rehabilitated HMA pavements

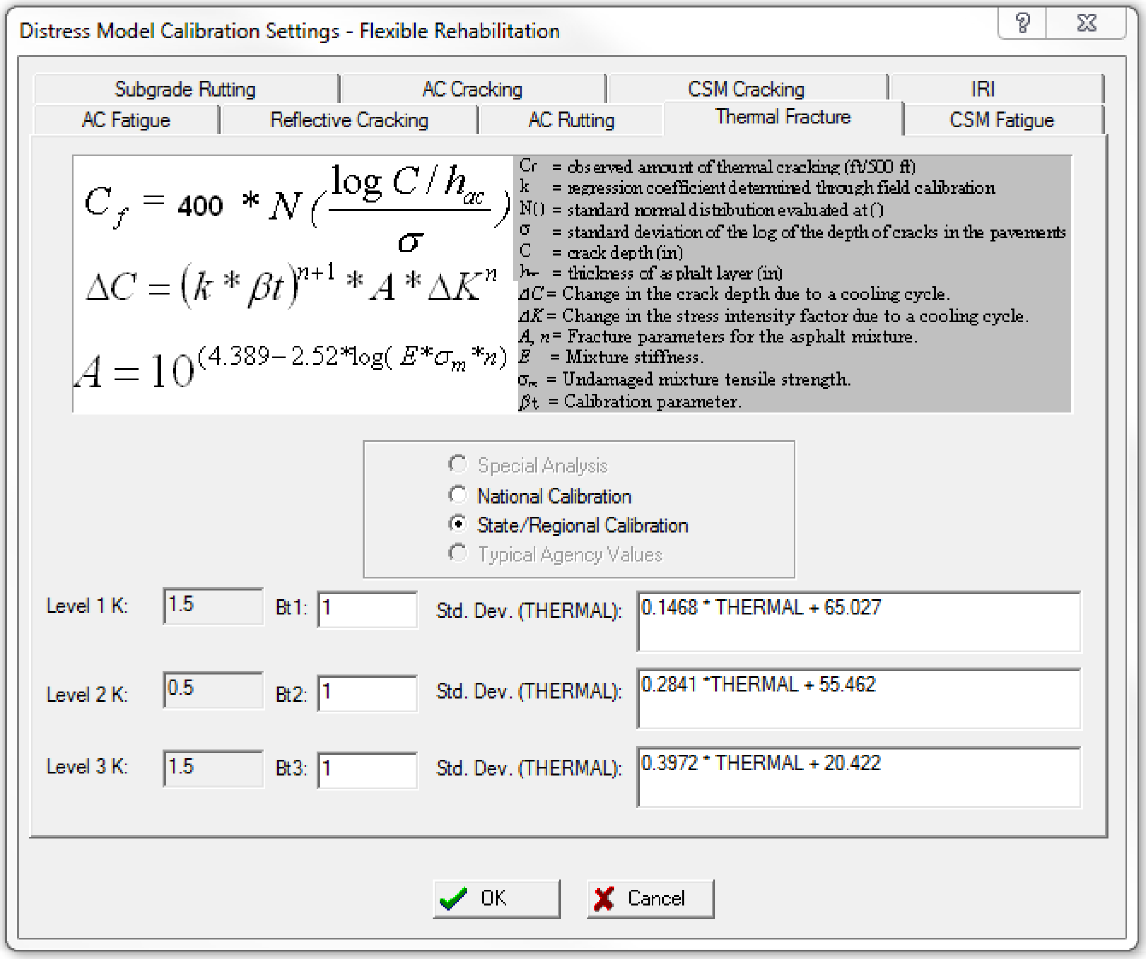


Figure B.17. Thermal fracture of rehabilitated HMA pavements

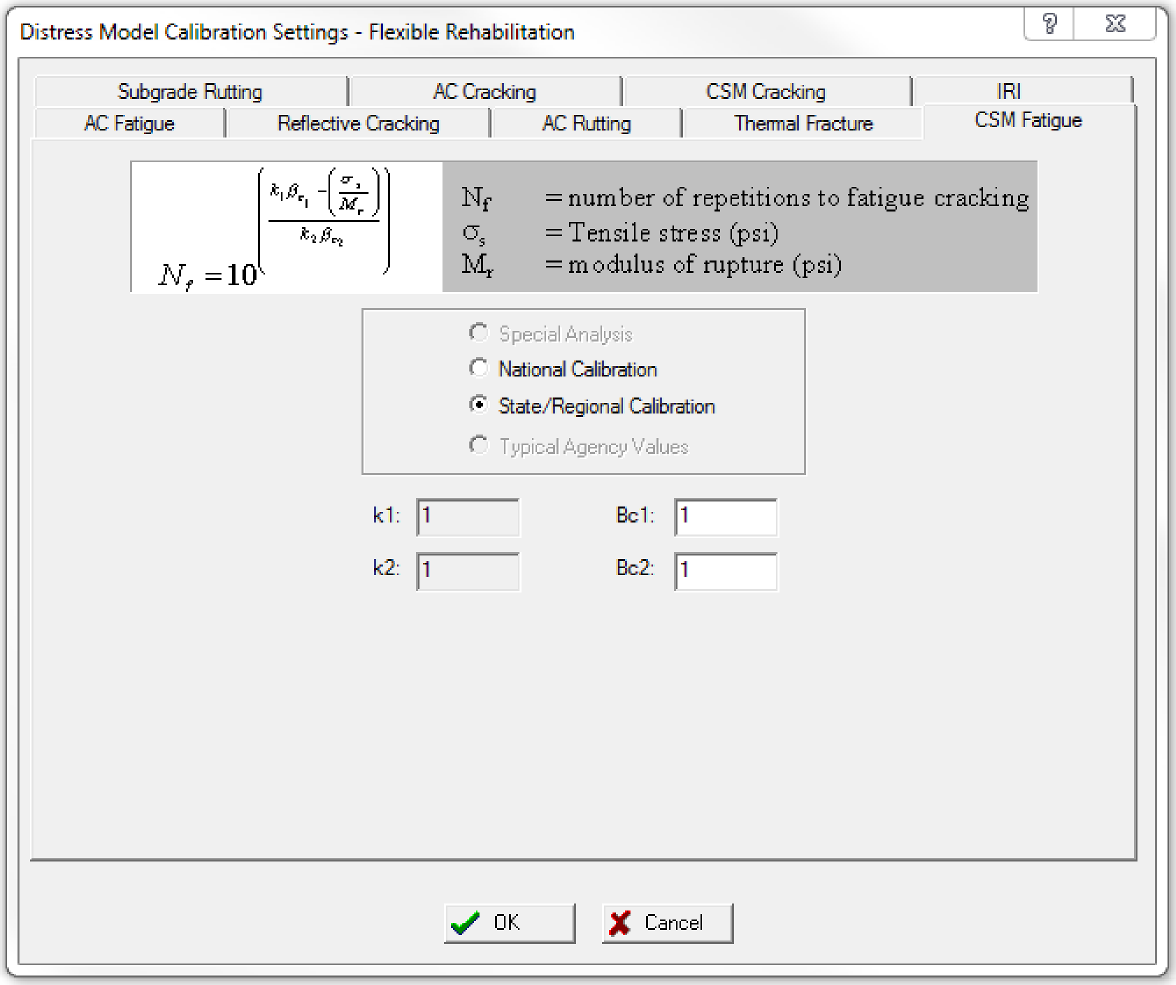


Figure B.18. CSM fatigue of rehabilitated HMA pavements

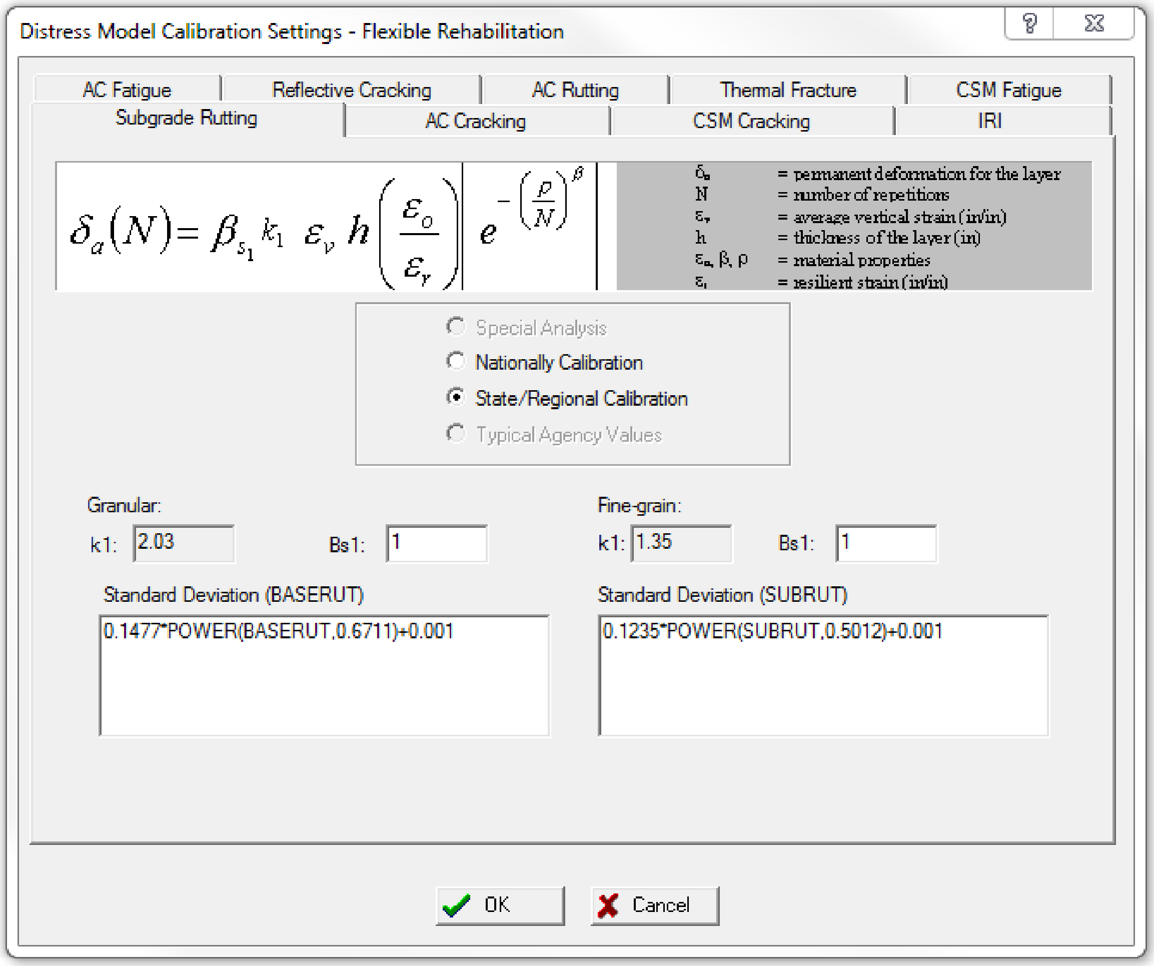


Figure B.19. Subgrade rutting of rehabilitated HMA pavements

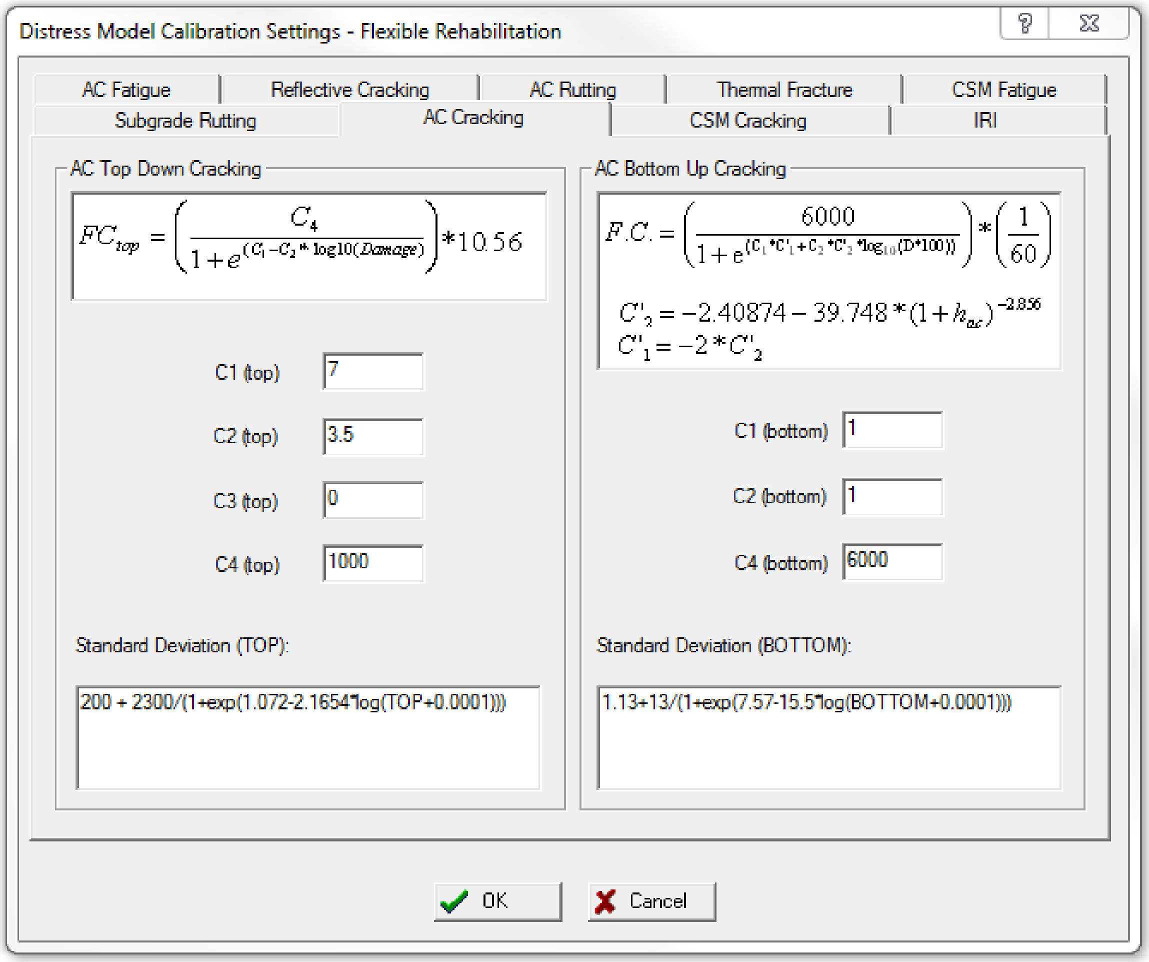


Figure B.20. AC cracking of rehabilitated HMA pavements

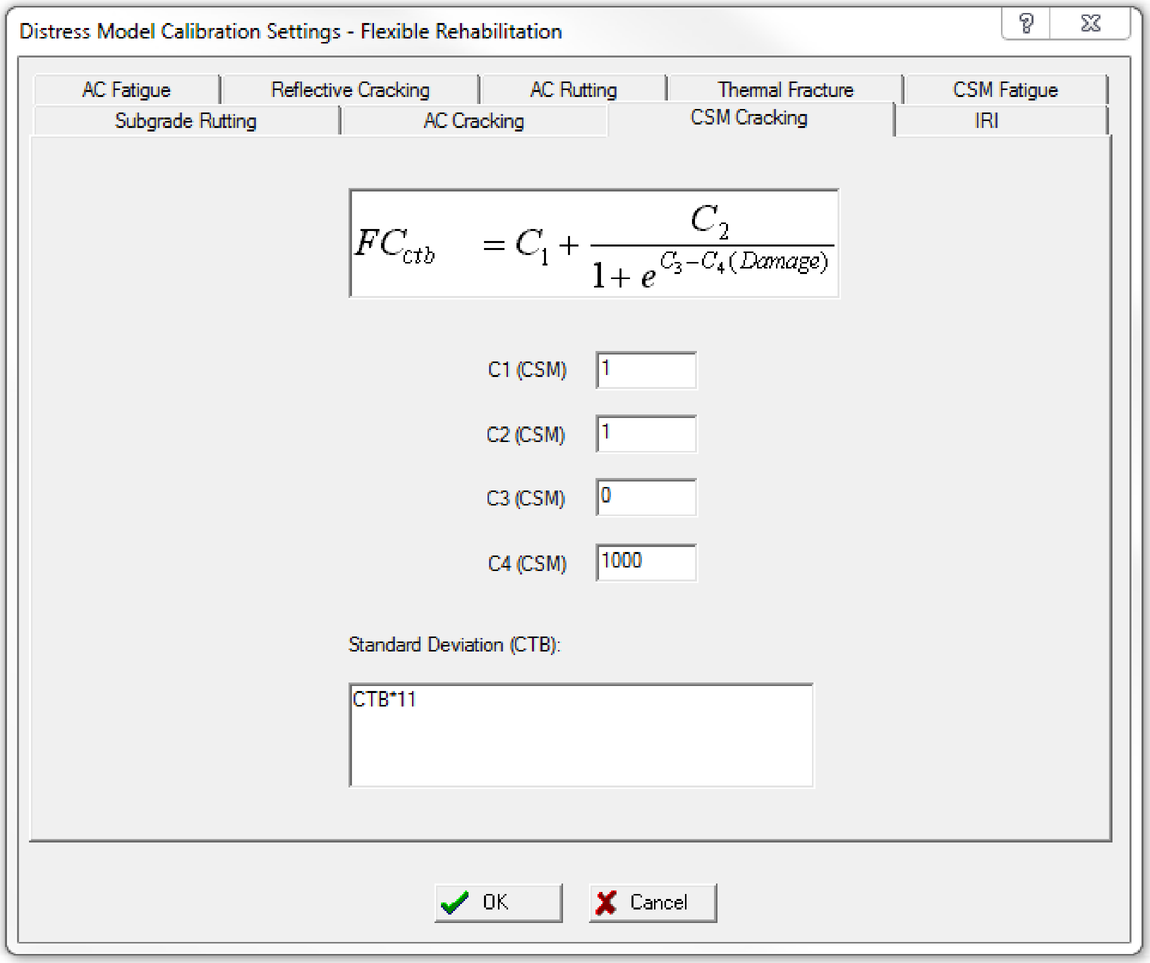


Figure B.21. CSM cracking of rehabilitated HMA pavements

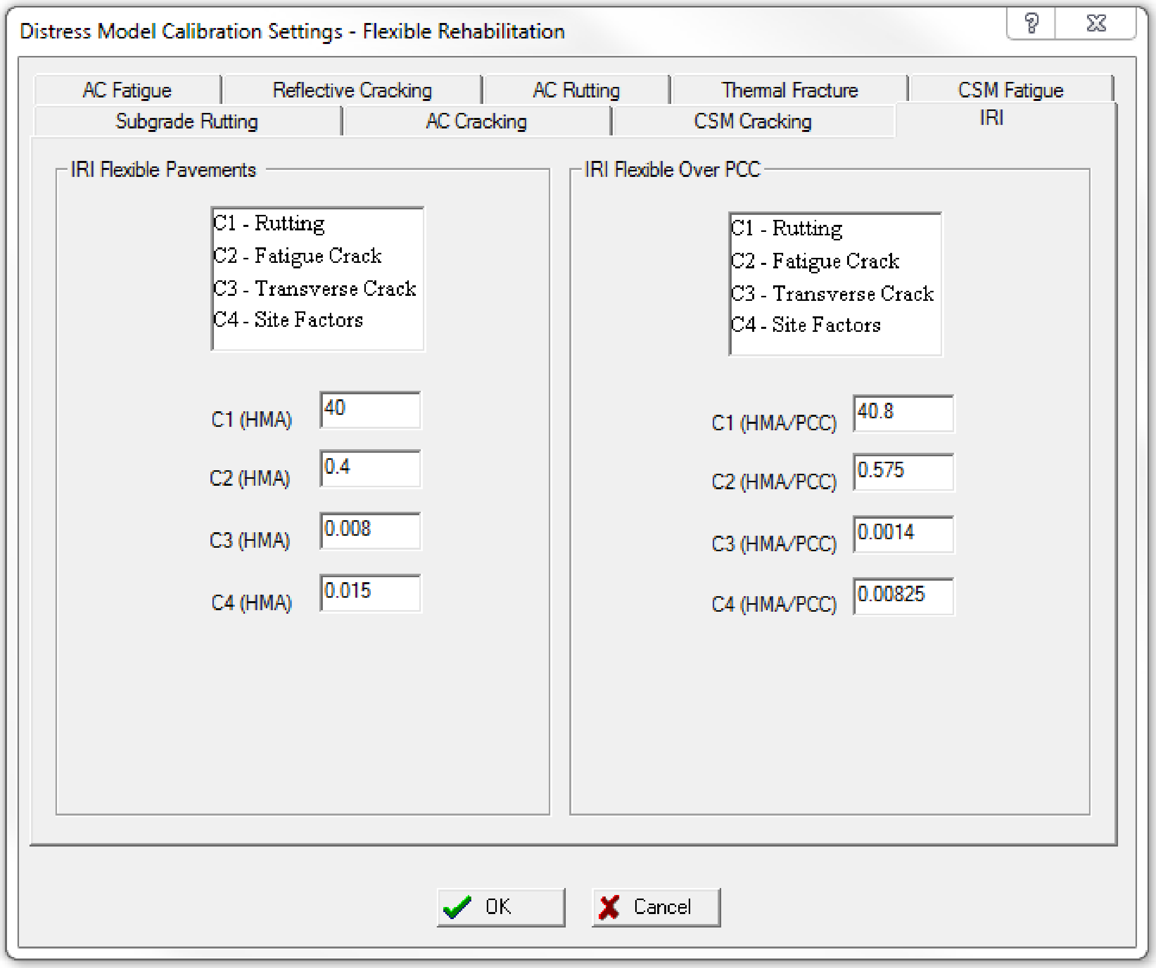


Figure B.22. IRI of rehabilitated HMA pavements